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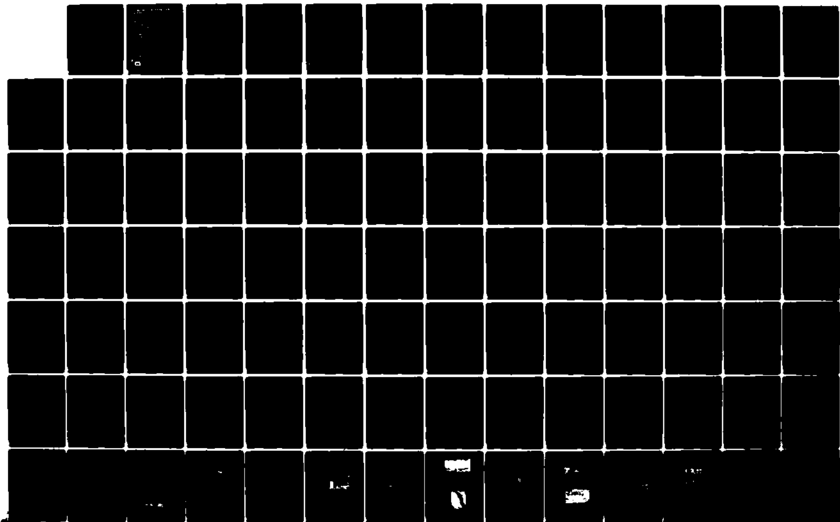

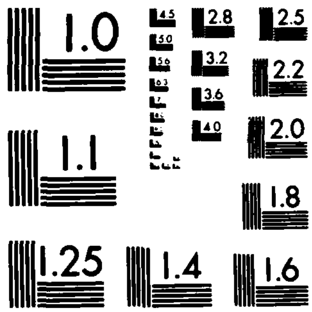

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CONSTRUCTION FOUNDATION REPORT

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AD A140034

MISSOURI RIVER
GARRISON DAM - LAKE SAKAKAWEA

VOLUME I
TEXT AND PHOTOS

NOVEMBER 1983

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Garrison Dam is located on the Missouri River about 70 miles north of Bismarck, North Dakota. This report summarizes foundation conditions encountered during the construction of Garrison Dam and associated structures such as the Snake Creek Embankment and protective works for the city of Williston. Items discussed in the report include:

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(Block 20)

(1) Site investigations
(2) Dam-site geology, including engineering characteristics of overburden
(3) Excavation procedures, including dewatering provisions and lignite excavations
(4) Cut-off trench and sheet piling cut-off
(5) Tunnels
(6) Foundation anchors
(7) Character of foundation
(8) Foundation treatment
(9) Instrumentation
(10) Potential problem areas

MISSOURI RIVER
GARRISON DAM - LAKE SAKAKAWEA
MISSOURI RIVER BASIN
NORTH DAKOTA

CONSTRUCTION FOUNDATION REPORT
VOLUME 1
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U. S. ARMY ENGINEER DISTRICT, OMAHA
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OMAHA, NEBRASKA
1982

GARRISON DAM - LAKE SAKAKAWEA
MISSOURI RIVER BASIN
NORTH DAKOTA

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CONSTRUCTION FOUNDATION REPORT

GARRISON DAM

MISSOURI RIVER BASIN, NORTH DAKOTA

PREFACE

Construction of the Garrison Dam was started in 1946 and all earthwork was essentially completed in 1955. Generation of power from the first generating unit began in 1956 and the fifth and final unit went on the line in 1960. Because of the extended period of time between the completion of construction and the preparation of this report, it has been necessary to rely to a great extent on information still remaining in the files at the Garrison Dam area office, as well as the design memos, construction plans and specifications, foundation reports, correspondence copies, and other pertinent data in the active files of the Omaha District Office.

There were no major foundation problems during construction of the Garrison Dam and most foundation work was completed in accordance with the contract design. A large amount of data was collected from the instrumentation made during construction and additional data continues to be compiled from subsequent monitoring. Minor erosion and seepage problems have developed since construction, but have not required extensive repairs or design changes. Although post-construction information is not normally included in the final foundation report, because of the extended period since construction and the availability of the information at this time, it is included herein.

CONSTRUCTION FOUNDATION REPORT
GARRISON DAM
MISSOURI RIVER BASIN, NORTH DAKOTA

CHAPTER 1 - INTRODUCTION

1. **LOCATION AND GENERAL DESCRIPTION OF PROJECT.** The Garrison Dam and Reservoir (Lake Sakakawea) is located on the Missouri River at river mile 1389.9 in McLean and Mercer Counties, North Dakota. The site is approximately 75 miles upstream from the city of Bismarck, North Dakota. Location and plan of the dam and reservoir are shown on Plate 1. An aerial view of the dam is shown on Photo 1. Pertinent data for the project is tabulated below:

PURPOSE: Multiple-purpose (flood control, irrigation, navigation and power).

DRAINAGE AREAS:

Missouri River Basin, sq. mile	529,350
Above Garrison Dam, sq. mile (includes 1,350 sq. mi. of non-contributing areas	180,940
Fort Peck to Garrison, sq. mile	123,215

RESERVOIR:

Maximum operating pool elevation, ft. msl	1,854
Minimum operating pool elevation, ft. msl	1,775
Pool area at maximum operating elevation, acres	410,000
Pool area at minimum operating elevation, acres	133,000
Length of pool, miles	200

RESERVOIR STORAGE ALLOCATIONS:

	Elevations msl	Capacity Acre-feet
Exclusive flood control	1854-1850	1,600,000
Flood control and multiple use	1850-1837.5	4,400,000
Carryover multiple	1837.5-1775	13,600,000
Inactive	1775-1673	4,900,000
Gross	1854-1673	24,500,000

DAM AND EMBANKMENT:

Top of dam, elevation, ft. msl	1,875
Length, less separate spillway, ft.	11,300
Damming height, ft.	180
Maximum height, ft.	210
Type of Fill	Rolled earth
Fill quantity, cu. yds.	66,500,000
Width at top, ft.	60
Width at base, ft.	2,600
Date of closure	15 April 1953

SPILLWAY:

Location	Left Abutment
Type	Concrete-line chute w/grated weir
Discharge capacity, cfs	827,000
Crest elevation, msl	1,825
Width, including piers, ft.	1,336
Gates, type	Tainter
Gates, number and size	28 - 40'x29'

OUTLET WORKS:

Number and size of conduits	1 - 26 ft. dia. 2 - 22 ft. dia.
Length of conduits, ft.	1,529
Service gates, type	Tainter
Service gates, size	18'x24.5'
Maximum discharge capacity	1 - 37,200 cfs 2 - 26,600 cfs

POWER FACILITIES:

Number of tunnels	5
Tunnel size	29 ft.
Penstock size	24 ft.
Penstock length	1,600 ft.
Power gates, number	10
Power gates, type	Vertical lift, "caterpillar"
Power gates, size	26' high by 12' wide
Ave. available gross head	154 ft.
Turbines, number	5
Turbines, type	Francis
Turbines, speed	90 r.p.m.
Surge tanks, number	2 per penstock
Surge tanks, diameter	65 ft.
Surge tanks, height	135 ft.
Plant capacity, total	400,000 KW
Dependable capacity	324,000 KW*
Ave. energy, million KWH	1,966*
Date, initial power generation	January 1956

*Values shown are average for the first 50 years.

2. CONSTRUCTION AUTHORITY. The Garrison Dam and Reservoir was authorized by the Flood Control Act approved 22 December 1944 (Public Law 534, 78th Congress, 2nd Session) as a part of the general comprehensive plan for flood control and other purposes in the Missouri River Basin.

3. PURPOSE OF REPORT. This report has been prepared in accordance with requirements set forth in Regulation No. ER 1110-1-1801, Engineering and Design Construction Foundation Reports, Office, Chief of Engineers dated 15 December 1981, change 1, dated 30 June 1982, and change 2, dated 1 April 1983. The report serves to record the conditions and engineering aspects of the various foundations for the structures comprising the Garrison Dam. Emphasis is given to geology, explorations, engineering characteristics of the materials, excavation methods, foundation treatment, instrumentation

and other foundation related subjects. These discussions are supplemented with appropriate drawings, boring logs, test data and photographs. Information for this report was taken from numerous reports in the holding files at the Garrison Dam area office and/or the files of the Omaha District Office. Detailed information concerning testing and foundation design data is too voluminous for inclusion in this report. However, all pertinent data is included.

4. CONTRACTORS AND CONTRACT SUPERVISION. Initial construction for the Garrison Dam commenced with the award of the contract for the access railroad in April 1946. Construction on the embankment began on 4 October 1947. The embankment was constructed in five stages. Closure on the embankment was made in April 1953 and the embankment was completed in April 1954. Most of the excavations for the powerhouse structure and associated intake and discharge channels, as well as the spillway and its channels, were accomplished under the five excavation stages. The eight tunnels and downstream portals were constructed under a separate contract commencing 22 April 1949 and concluding 31 May 1951. A portion of Tunnel No. 4 was mined under the Stage 1 contract as a test tunnel to furnish technical information to confirm the design of the main tunnels. The test tunnel also afforded bidders on the main tunnel contract an opportunity to assess construction problems. Construction on the spillway structure and gates began 17 June 1952 and was completed 7 October 1955 and construction on the spillway stilling basin began 1 March 1955 and was completed 12 July 1957. The intake structure construction began on 16 September 1949 and was completed on 17 June 1954, and construction on the powerhouse structure began 7 July 1953 and was completed on 5 December 1956.

4.1 The Snake Creek Embankment, which was constructed across an arm of the Garrison Reservoir to serve as relocation for a highway and railroad, as well as to serve as a sub-impoundment for diversion of water into the Missouri River Diversion project, was constructed during 1951 and 1952 under one contract. No major modifications or additions have been made to the project since completion. The principal contractors for construction of the Garrison Dam project were as follows:

CONTRACT	CONTRACTOR	APPROXIMATE AMOUNT OF CONTRACT
W-32-015-ENG-119, Excavation and Main Embankment, STAGE I - Excavation of Powerhouse Structure, intake and outlet channel, cut-off trench and sheet piling cut-off. Embankment fill (partial) west of river.	H. N. Rogers & Sons Co. & Forcum-James Co. & S. K. Jones Const. Co.	$ 6,349,800 AWARD DATE OF CONTRACT 8 Sep 47
W-32-015-ENG-530, Excavation & Main Embankment, STAGE II - Excavation of powerhouse intake channel, spillway and embankment toe drain trench. Embankment fill (partial) east of river.	Peter Kiewit Sons Co. & Morrison-Knudsen	$13,622,000 23 Sep 48
W-32-015-ENG-1212 Outlet works tunnels	S. A. Healy Co. & Material Service Corp.	$15,218,000 9 Mar 49
DA-32-015-ENG-66 Outlet works intake structure	Peter Kiewit Sons Co. & Morrison-Knudsen Co.	$15,125,800 31 Aug 49
DA-32-015-ENG-510, Excavation & Main Embankment, STAGE III - Excavation of powerhouse intake and outlet channels, borrow area A-9, complete embankment fill west of river.	Peter Kiewit Sons Co.	$ 5,740,500 3 Nov 49
DA-32-015-ENG-749 Outlet works stilling basin and powerhouse foundations	Unite Construction Co.	$ 9,749,200 22 May 50
DA-32-015-ENG-1416 Snake Creek Embankment	Lytle & Greene	$ 2,591,600 28 May 51
DA-32-015-ENG-1551 Outlet works regulat' gates	Consolidated Western Steel	$ 1,088,000 13 July 51

CONTRACT	CONTRACTOR	APPROXIMATE AMOUNT OF CONTRACT
DA-32-015-ENG-2770 Outlet works, Penstock and surge tanks	Southwest Welding & Mfg. Co.	$ 6,624,800 30 Jun 53
DA-32-015-ENG-2634 Embankment Relief Wells	Layne-Minnesota Co.	$ 124,900 16 Mar 53
DA-32-015-ENG-2910 Excavation & Main Embankment, STAGE V - Excavation of remaining spillway approach & return channels, borrow areas C, D & G and completion of embankment toe drain at closure area. Complete embankment fill in closure area and east of closure	Peter Kiewit Sons Co. & Morrison-Knudsen Co.	$11,592,900 28 Oct 53
DA-32-015-ENG-3945 Spillway Stilling Basin	Peter Kiewit Sons Co. & Morrison-Knudsen Co.	$ 6,949,800 23 Feb 55
DA-015-CIVENG-57-550 Outlet works penstocks and surge tanks 4 & 5	Southwest Welding & Mfg. Co.	$ 7,555,400 30 Apr 57

Contract supervision and administration was by the Garrison District Corps of Engineers, which was located at the Garrison Damsite.

CHAPTER 2 - FOUNDATION EXPLORATIONS AND OTHER RELATED STUDIES

1. INVESTIGATIONS PRIOR TO CONSTRUCTION.

1.1 ALTERNATE SITES. Several damsites were given consideration prior to selection of the project site. The requirement imposed on the selection of a damsite considered that storage capabilities be the maximum possible and the dam be constructed on a safe foundation within reasonable costs. The downstream limit for the damsite was the city of Bismarck, North Dakota, and the upstream limit was a location at which the upper reaches of the reservoir would not inundate the city of Williston, North Dakota. First consideration was given to the Price site located near the town of Price about 24 miles above Bismarck. Using the same pool levels as for Garrison Dam, this site would have given more storage than the Garrison site, but reconnaissance studies did not reveal a feasible location for a spillway without excessive excavations. It also would have caused excessive flooding of the Knife River Valley and would have required many miles of railroad relocations, as well as relocation of several small towns. Another site given consideration was located about 10 miles downstream from the Garrison site. This was known as the Stanton site. Using the same pool levels as Garrison Dam, this site would have provided more storage than Garrison Dam, but the required embankment yardage far exceeded that of the Garrison site and cost of the additional storage could not be justified. A total of five borings were made at the site. These explorations revealed similar foundation conditions as encountered at the Garrison site. In addition to the Price site and the Stanton site, five sites above Garrison and below the mouth of the White River were given cursory investigations but did not provide sufficient storage for development of the Missouri River Basin.

1.2 SELECTED SITE.

1.2.1 Boring Investigations. Initial subsurface investigations for the Garrison Dam were completed in 1931 by the Kansas City District. These investigations consisted of 21 wash borings to investigate the overburden. Where bedrock was encountered, 3-inch diameter core holes were continued into the bedrock. Two large test pits were also excavated. The results of these investigations were published in House Document No. 238, 73rd Congress, 2nd Session. Interpretation of this subsurface work was made by Professor Warren J. Mead, a consulting geologist.

1.2.2 Investigation for the Survey Report commenced in 1943 at which time 128 borings and 1 test pit were completed at the damsite. In 1945, borings were made for the Definite Project Report, including 27 borings in the conduit area, 56 borings in the right abutment (which during earlier studies was considered as a spillway location), 45 borings in the left abutment (final spillway location), 57 borings in the embankment foundation, and 128 borings for borrow materials. Churn drills and rotary drills were used to recover samples. The overburden was generally sampled with a hand auger or churn drill, which recovered 6-inch drive samples. The bedrock was sampled with a rotary drill. Samples in the soft materials were collected by

jacking sample tubes with metal liners, or where the materials could be cored, the samples were collected by double tube core barrels that retrieved a 5-3/4-inch diameter core. These methods were compatable with all types of materials encountered and yielded satisfactory samples.

1.2.3 Following the investigations for the Definite Project Report more than 250 additional borings were completed, most of which were for borrow material studies. About 65 were used for exploring the originally proposed right (west) spillway location and a cut-and-cover outlet work. Both of these plans were abandoned. Listed below is a tabulation of the number of borings completed at the damsite as of May 1948, at which time the Analysis of Design, Excavations and Main Embankment was published:

TABLE 1

Location	No. of Borings	
Spillway and East Abutment	65	
East Bank Floodplain	41	
Island and River Channel (includes construction bridge)	41	
West Bank Terrace	90	
West Abutment	15	
Outlet Works (includes intake channel, tunnels & tailrace)	80	
		332
Borrow Area A (west bank)	75	
Borrow Area B (east bank)	86	
Borrow Area on East Bank Floodplain	20	
Silt Blanket Studies	13	
Miscellaneous Borrow	23	
		217
Cut and Cover Outlet Works (abandoned)	15	
West Abutment Spillway (abandoned)	50	
		65
Total:		614

In addition to the 614 borings listed above, at least an additional 50 borings were completed for the Spillway Design Memorandum studies. A reasonable estimate for the total number of investigational borings completed at the Garrison damsite would be well in excess of 800 borings. See Plate 2 for location of borings and Plates 3 through 39 for boring logs and sections. Geologic profiles for the embankment, spillway, and powerhouse areas are shown on Plates 40 through 43.

1.2.4 **Plate Bearing Test**. Prior to construction of the powerhouse, a plate bearing test was completed in Hole 608 for the purpose of determining the modulus of elasticity (E) of the Fort Union foundation bedrock. See Plate 44 for location of test and log of hole. The test served to

compare the in-place conditions with the laboratory tests and was also used for settlement analysis of the outlet works structures. The test also determined E at different depths under the influence of a confining surcharge due to overburden load. The tests were conducted at four elevations in the Fort Union Formation which consisted of a compaction type clay-shale interspersed with lignite beds. The shale had a N-S and E-W system of usually tight and nearly vertical joints. The lignites carried fairly strong flows (30 to 70 gpm) of ground water through numerous joints and fractures. The hole was drilled with a 39-inch diameter Sanborn earth auger powered by an Ingersol Rand drill. A 39-inch diameter calyx drill using steel shot was employed to penetrate the lignite and limestone, when encountered. Collapsible Armco galvanized corrugated pipe (39-inch inside diameter) was used to case the hole. The test equipment and arrangement are shown on Plate 45 and Photo 2. Two types of loadings were used in four bearing tests: 1) A fast test - in which load increments were applied at the rate of about 2 tons per square foot per 2 minutes, and 2) A slow test - in which it was intended to maintain the load increments until full adjustments were reached, as determined by the semi-log plot of time vs. settlement, showing that the test had been carried into the secondary compression stage. A detailed analysis of this testing is covered in "Report of Plate Bearing Test Hole 608," Garrison Dam, December 1947, which is available from the F&M Branch of the Omaha District. The results of the testing showed some loading effects, but were not considered too reliable. The original purpose of determining E of the Fort Union Formation and verifying with laboratory testing was not realized. The bearing test was lengthy and costly requiring 1-1/2 years to complete and costing approximately $55,000.00.

1.2.5 **Pumping Test BB Lignite Hole 608.** After completion of the bearing plate test in Hole 608, a pumping test (referred to as Pumping Test No. 1) was conducted to determine the direction of flow and coefficient of permeability of the BB lignite. It was hoped that the test would show whether or not the 1A lignite was hydraulically connected to the BB lignite. See Plate 40 for locations of lignite beds. Prior to running the pumping test in Hole 608, test piezometers were installed in adjacent Holes 709, 711, 741 (BB lignite), and Holes 708 and 746 (1A lignite). In addition, observation wells were installed in Holes 701, 702, 703, 704 and 706. Several observation wells were also drilled specifically for the pumping test. Before beginning the testing, lignite BB was drilled through and the hole was pumped for a considerable period to clean the hole of any grout that had leaked into the lignite during setting of the bearing test plate. Prior to the pumping tests, the hole was not pumped for 5 days to allow the establishment of a static piezometric surface. The equipment used in the test consisted of an Ingersol Rand Model No. 35 high head sump pump, a LeRoi portable compressor, and a 500-gallon water tank. Using both the equilibrium and the non-equilibrium methods of determining coefficient of permeability, as described in Geologic Survey Water Supply Paper 887, "Methods for Determining Permeability of Waterbearing Materials," the following permeabilities were established:

TABLE 2

Apparent Coefficient of Permeability of Lignite at 60° F.

Method of Computation	Units of ft/min	Units of cm/sec
Equilibrium	0.216	1100×10^{-4}
Equilibrium	0.185	940×10^{-4}
Non-equilibrium	0.184	930×10^{-4}
Non-equilibrium	0.171	820×10^{-4}

Note that the methods of computation employed were developed for homogeneous, porous media, whereas the water in lignite is actually carried in a system of joints and fractures.

1.2.6 The results of the testing showed the BB lignite was very pervious, suggesting the joints and fracture to be continuous for a considerable distance. Since the piezometers installed in the 1A lignite did not fluctuate significantly, and the piezometers in the BB lignite fluctuated greatly, it was concluded the two lignites were not hydraulically connected.

1.2.7 **Frozen Plug Sampling.** Undisturbed samples of river valley sediments were obtained by frozen plug methods in connection with studies to determine permeability and density of foundation sands. These data were to be used for the design of relief wells and as an aid in evaluating pumping tests. The equipment for the sampling was obtained from the Waterways Experiment Station, Vicksburg, Mississippi and consisted of an alcohol-circulating freezing unit and pump with hoses, 2-7/8-inch inside diameter Shelby tube sample container, two 2-7/8-inch diameter pistons with piston rods, a sub for connecting Shelby tube to drill rod, and appropriate annular and cleanout augers. The sampling was conducted under the supervision of WES personnel. Samples were obtained by the Shelby tube push sampler which was overdrilled with an annular auger after which the auger was withdrawn and a plug at the bottom of the sample in the Shelby tube was frozen by a freezing chamber. The freezing chamber was lowered into the hole over the drill rods until it reached about 0.1 foot from the bottom of the sample tube. Cold alcohol was then circulated until the bottom of the sample had been frozen (usually 15 to 45 minutes), whereafter the drill rod with attached sampler and freezer was withdrawn as a unit. See Photos 3 and 4 for freezing apparatus.

1.2.8 Two holes, Drill Holes 781 and 800, were sampled by the frozen plug method to obtain undisturbed samples for permeability and density testing. These borings were located in the central portion of the river valley along the relief well alignment. See Plate 2 for location of borings. The results of the permeability tests conducted on the undisturbed samples showed that horizontal permeabilities were somewhat higher than the vertical permeabilities. The tests were considered to be reliable, with the most reliable samples coming from Drill Hole 800. A detailed report describing the sampling and density-permeability testing procedures is included in

Laboratory Branch Report, Test Report 5, Garrison Dam, "Undisturbed Sampling of Sediments by the Frozen Plug Method," February 1949. The report is available from the F&M Branch files of the Omaha District.

1.2.9 **Study of Natural Slopes of the Fort Union Formation.** A study was made of the natural slopes of the Fort Union Formation for guidance in developing stability analyses to determine safe slopes for deep excavations. Numerous field observations were made and profiles were measured of existing natural slopes, man-made slopes in strip mines, and excavated railroad cuts. The study was accomplished by transit and stadia mapping. The natural slopes were mapped along the Missouri River in the vicinity of the damsite. The largest railroad slope was about 40 feet high and 2,000 feet long cut to a 1 vertical on 1 horizontal slope. Cuts in the lignite mines ranged from 30 to 60 feet high with slopes of 1 vertical on 1/2 horizontal to 1 vertical on 1 horizontal. The results of this study led to the adoption of shear strength values of C = 0.70 tons per square foot and $\phi = 20^{\circ}$ or tan $\phi = 0.36$. A slope chart is shown on Plate 46.

1.2.10 **Test Driving of Steel Sheet Piling.** Prior to excavation at the damsite, the Contractor for Stage I and the Corps of Engineers made a supplemental agreement for the Contractor to test drive sheet steel piling under requirements similar to those required in the Stage I specifications. The Contractor was directed to use the same equipment and procedures as he intended to use on the construction of the sheet piling cut-off wall. The Government furnished the steel sheet piling which consisted of 36 piling sheets 35 feet long. The site selected for the testing was at Drill Hole 473. The ground surface elevation was 1694.1 and the trench from which the sheet piling was driven was elevation 1683.3. The top of the Fort Union Formation to which the sheet piling was driven was elevation 1593.1. The driving operations commenced 14 May 1948 and continued through 22 May 1948. The sheet piling was driven with a Marion Crane Model 362 which handled a 9B-3 double-action steam hammer with fish-tail leads and anvil-type driving block. The sheet piles were driven in pairs with the blows per minute averaging from 102 to 106. The resistance at 15-foot depth averaged 1-1/2 blows per inch and at a 32-foot depth averaged three blows per inch. Several cut-offs were required because of bucking of the top from driving operations. The driving time for the first 35-foot section was one hour and ten minutes. A tabulation of the sheet pile driving is shown below:

TABLE 3
FINAL DRIVING DATA

Pile	Cut-off Elev.	Grd. Elev.	Trench Elev.	Point Elev.	Effective Length	Ft. Union Elev.	Penetrate Ft. Union	Final Blow/ Min.	Final Blow/ Inch
1	1690.2	1694.0	1683.3	1587.3	101.0	1593.0	5.7	106	20
2	1690.2	1694.0	1688.3	1587.3	101.0	1593.0	5.7	101	20
3	1690.3	1694.1	1688.3	1590.3	98.0	1593.0	2.7	98	19
4	1690.3	1693.9	1688.3	1590.3	98.0	1593.0	2.7	98	19
5	1692.1	1693.7	1688.3	1589.8	98.5	1593.0	3.2	96	54
6	1692.1	1693.8	1688.3	1589.1	99.2	1593.0	3.9	96	54
7	1691.1	1693.9	1688.3	1589.8	98.5	1583.0	3.2	102	30
8	1691.1	1694.0	1688.3	1590.0	98.3	1583.0	3.0	102	30
9	1690.3	1694.0	1688.3	1588.4	99.9	1593.0	4.6	106	29
10	1690.3	1694.0	1688.3	1588.4	99.9	1593.0	4.6	106	29
11	1690.0	1694.2	1688.3	1587.3	101.0	1593.0	5.7	98	16
12	1690.1	1694.2	1688.3	1587.1	101.2	1593.0	5.9	98	16

PILE LENGTHS AND CUT-OFFS

Pile	No. of Sec.	Lgth. Per Sec.	Total Lgth.	No. of Cut-offs	Total Cut-off Lgths.
1	3	35	105	3	2.1
2	3	35	105	3	2.1
3	3	35	105	6	5.02
4	3	35	105	6	5.02
5	3	35	105	3	2.65
6	3	35	105	3	1.95
7	3	35	105	4	3.72
8	3	35	105	4	3.88
9	3	35	105	4	3.07
10	3	35	105	3	3.03
11	3	35	105	4	2.24
12	3	35	105	4	2.03
TOTAL	36	1,260	1,260	47	36.81

Original length of each pile - 105.0 feet
Total effective length of pile driven - 1,223.19 feet

Extraction of the piling for observation of the physical condition of each sheet began 25 May 1948 and was completed on 15 June 1948. All of the piling was found to be in good condition except Pile No. 6 which had 4.6 feet of its bottom distorted into a wavy shape with the lock on one side of the pile broken for about 4.6 feet. See Photos 5 and 6.

1.2.11 **Anchor Pullout Test.** Pullout tests were conducted in the right (west) abutment in the Fort Union Formation during 1946 on anchors set in five different types of anchor holes. Four anchors had bulbs belled out to diameters of 12, 18, 24 and 32 inches and one anchor had no bulb. The bulb types were subjected to four cycles of loading. For the first three cycles the load was carried to 25 tons and released. For the fourth cycle the load was carried to 35 tons, or failure, and released. The fourth cycle results were plotted as shown on Curves 1 through 4 on Plate 47. The straight (no bulb) anchor was not subjected to cyclic loading but was loaded to failure. The results are plotted on Curve No. 5. The design of the anchors is also shown on Plate 47. The type of concrete and mix used in the bulb is not known, but assumed to have been made with a bank run gravel. The results of the tests suggested that plastic flow of the Fort Union Formation would take place at about 35,000 pounds per square foot and would cause initial failure of the anchor. It was concluded from the tests that 25,000 pounds per square foot would be a reasonable upper limit and this value was used in the spillway anchor designs for the allowable contact soil pressure on the horizontal projection of the bulb.

1.2.12 **Test Grouting of the Lignite Beds.** Prior to construction of the grout curtains at the damsite, a grout test panel using neat cement and a Stop Method of grouting, was constructed by hired labor in the west abutment powerhouse area. The grouting was accomplished in the 1A and BB lignite beds during 12 to 28 July 1948. After grouting by the Stop Method, the grouted area was excavated and the effectiveness of the grouting was observed by examination and mapping of the grouted joints and fractures. The results of this study are covered in detail in the report, "Supplement to Report No. 5 - Test Grouting," which is available from the Omaha District, F&M Branch files. In order to clearly observe the extent to which the grout had penetrated the joints of the lignite, each bed was uncovered and swept by hand, after which a coal saw was employed to make vertical cuts coinciding with the line of the grout holes. See Photos 7 and 8 for top of lignite bed and vertical cuts showing grouted joints. The results of this study indicated the lignite joints and fractures could be grouted with relative ease, but high pressures and thin grout mixes would not be applicable, as such procedures would result in an uneconomical use of grout by forcing it long distances away from the curtain. It was also noted that by using a packer and employing the Stop Method of grouting, instead of the Full Depth Method, the amount of grout placed could be better controlled using applicable water-cement ratios and pressures as determined from the pressure testing of each lignite horizon. It was also concluded that a spacing of 20 feet should prove satisfactory in obtaining an overlap between grout holes. This conclusion was based on the fact that a minimum effective radius of grout filling of 10 feet per hole was obtained in the BB lignite in three of the four test

holes. As the horizontal bedding planes in the lignite beds were observed to serve as grout channels in the same manner as the near vertical joints, it was also concluded that vertical grout holes would serve to distribute the grout as satisfactorily as angle holes. Grout mixes and grout takes will be discussed in subsequent paragraphs concerned with the lignite grouting. The drilling equipment, grouting equipment, packers, and core sample of grout filled lignite fracture are shown on Photo 9.

2. INVESTIGATIONS DURING CONSTRUCTION.

2.1 **LIGNITE MINING AND STORAGE STUDY.** Prior to major excavations at the damsite, it was recognized that consideration was required as to how the approximately 4-1/2 million tons of lignite included in the required excavations would be handled. It had been determined that all beds 3.0 feet and over in thickness would be salvageable. With this objective in mind, during the summer of 1947, an extensive inventory was made of several strip mines in the state to investigate mining and storage methods. Pertinent comments from this study are as follows: 1) No particular difficulty was observed in shovel and dragline operations in local mines. Limestone horizons of the magnitude of 2 feet were handled by average sized equipment, and the larger sized shovels and draglines could handle heavier layers. Thicker horizons of limestone generally required shooting. The mud capping method of blasting was usually used in the strip mines. 2) Scrapers need a pusher "Cat" for loading Fort Union and a ripper to break up the material to be loaded. A 3- to 6-inch bite of the scraper blade into the Fort Union loaded most expediently and kept the maximum dimension of scraper spread material in the range of a foot. 3) Lignitic coal in place may vary considerably in hardness and toughness due to moisture content and inherent structure. Depending upon whether the lignite is processed through a tipple, the commercial sizes desired, and the state of the coal in place, various degrees of shooting or no shooting are practiced at the mines. To a certain extent, the control of the coal size in the pit can be regulated by shooting. Frozen lignite and overburden may require shooting. 4) Storage of lignite in any quantity is not practiced at the local mines. An exception to this was a pile of slack (screenings) which had been accumulating at a mine for over 20 years. The pile contained about 200,000 tons. Every summer, two or three surface fires developed on this pile due to spontaneous combustion. The fires were extinguished by a small amount of water brought to the pile by a 1-1/2-inch pipe. It was determined that piles of any size of lignite in the open will heat and ignite. The solution appeared to be to cut off the circulation of air through the lignite pile by storage under water, in underground pits, or protective earth dikes and caps on piles aboveground. As lignite exposed to air readily breaks down to a fine size, the natural weathering of the material aids in reducing the circulation of air in the pile. Information from the study proved useful in developing a method of stockpiling the Garrison Dam lignite. Lignite storage will be discussed in a subsequent paragraph.

2.2 WEATHERING STUDY OF SELECTED ROCK FROM THE FORT UNION. During the summer of 1948, a study was made of the Fort Union sandstone and limestone members that could be salvaged from required excavation and used in the closure dike. This cemented rock was divided into four classifications based on physical characteristics and composition. The rocks included concretionary limestone, shaly limestone, sandstone and concretionary sandstone. The limestone concretions were large, varying from 6 to 10 feet in diameter. The sandstones and limestones were bedded, and the sandstone concretions were the hardest rock excavated at the damsite. To compare rock exposed to weathering with protected rock, a site for the test was selected that permitted part of the samples to be placed under water. Six samples of each rock type were taken from rock salvage piles, weighed and photographed (see Photos 10 and 11), and each sample carefully checked to determine if any fractures existed that might affect the results of the test. Each rock was inspected at intervals and pertinent information was recorded. Following a period of one year, there was no deterioration of any of the rock types placed under water. It was noted, in the case of the rock types exposed to the air, that the sandstone concretions did not show any weathering and the bedded sandstone rock was only superficially damaged. However, all the limestone samples were completely disintegrated. From these observations, it was concluded that the concretionary sandstone and bedded sandstone were the most resistant to weathering and would be suitable for use in the embankment closure. The limestone, although subject to severe weathering when exposed for a long period, would be adequate for closure if exposed for only a short period.

2.3 INVESTIGATION OF WATER LOSSES IN STAGE I EMBANKMENT IMPERVIOUS ZONE. During the drilling of grout Hole No. 1, west abutment grout curtain, it was noted that a small water loss occurred in the embankment at a depth of 20 feet and a large water loss occurred at a depth of 35 feet. Similar water losses were noted when adjacent grout Hole No. 2 was drilled. This was accompanied by a water interchange between the holes. Location of the west abutment grout curtain is shown on Plate 48. To investigate this condition, several 36-inch diameter auger holes were drilled in the area for visual inspection, and exploratory churn drill holes were drilled in adjacent areas. The churn drill holes were kept filled with water and observations were made of the auger holes to note if water seeped laterally from the churn holes into the auger holes. The results of this testing did not reveal any water seeping through the embankment fill into the auger holes. In-place inspection was made of the embankment fill as encountered in the auger holes and density samples were taken to determine the in-place density of the fill. A thin, pervious clayey gravel layer was observed in the auger holes at about elevation 1741, which was in the general area of the water losses. This clayey gravel extended from Station 28+20 west for about 200 feet. Visual inspection showed that the junction of the 1948 season of fill placement with the 1949 season was at about elevation 1740. This junction was well bonded and was not a passageway for any water movement. It was concluded that the water losses encountered in the grout holes were so localized that they would not require any special embankment modifications. However, the study results proposed that remedial treatment in the fill be attempted by using a thin cement grout at low pressure (10-15 psi), which would be placed by grouting

away from Hole No. 1 in successive holes toward an exploratory churn drill hole approximately 50 feet up station. This hole was observed to be in communication with grout Hole No. 1. By grouting away from grout Hole No. 1, it was hoped to force the grout to emerge through any water passage leading to the adjacent churn drill hole.

2.4 **INVESTIGATION OF VERTICAL GROUT FILLED CRACK IN STAGE I EMBANKMENT.** During the grouting of the west abutment grout curtain through the Stage I embankment, July 1949, it was noted that grout was flowing from the embankment at a location 6 to 10 feet east of grout Hole No. 24. It was further noted that in backfilling grout Hole No. 22, 30 cubic feet of grout was needed to fill the hole, which was a much larger quantity than normal to backfill a grout hole of this size. To determine the path the grout in Hole No. 22 was taking, exploratory borings were made in the embankment with a Buda Auger drill. As the grout hole was cased through the embankment, it was assumed the grout was coming up on the outside of the casing and venting through the embankment. The vented grout had formed a grout filled vertical crack in the embankment that passed through the grout hole and paralleled the centerline of the underlying cutoff trench. The results of the investigation did not develop a definite conclusion for the vertical crack in the embankment, but it was thought that because the grout hole was drilled in an area of possible high embankment stress, due to differential settlement between the natural foundation and the backfill in the cut-off trench, the high stress condition may have contributed to the crack. However, it concluded that the large backfilling requirement of grout Hole No. 22 was a result of the backfill grout flowing into a gravel layer intersected by the underlying cut-off trench.

2.5 **MOVEMENT IN POWERHOUSE STRUCTURE FOUNDATION.** During excavations for the upstream and downstream powerhouse shear keys, several 4-inch wide saw cuts were made with a coal saw to facilitate shovel excavations of the keys. For location of keys see Plate 49. On 11 September 1950, movement of the foundation bedrock was noted in the saw cuts. To permit visual inspection of the foundation in these movement areas, six large diameter auger borings, located as shown on Plate 50, were drilled and detailed mapping and observations were made. Excerpts from Investigation Report No. 12, Soils and Geology Branch, Garrison District, "Movement in Powerhouse Structure Foundation," 11 October 1950 are as follows:

2.5.1 The first indication that movement of the powerhouse structure foundation had occurred was noted in saw cut No. 5. Originally, the width of the cut was about 4 inches, but the day after its completion, 15 September 1950, the west end of the cut had closed. Examination revealed that a maximum movement of 3 inches had occurred during the 24-hour period since completion of the saw cut. Further evidence of foundation movement was noted in the upstream key excavation which was completed on the same day as saw cut No. 5 (see Plate 51). Along the downstream face of this key, a roughly horizontal crack had formed which ran from a point 60 feet west of the centerline to Tunnel No. 4 eastward 130 feet as shown on Plate 52. That part of the foundation above the crack appeared to have moved into the key

excavation and overhung the lower part, as shown in section on Plate 53. Initial measurements of the overhang ranged between 0.02 and 0.03 feet. During detailed study of the crack on 18 September 1950, the overhang had increased and averaged about 0.06 feet with a maximum of 0.11 feet, as shown on Plate 52. The crack followed along a fat clay lens 2 to 4 inches thick lying between two silty clay zones containing numerous thin silt laminae, except in the reach 50 to 70 feet east of the centerline where it angled steeply toward the base of the key excavation and followed a thin lignite stringer. In addition to the crack, another result of the foundation movement, as observed in the upstream key, was the formation of the loose fragmented zone in the fat clay which was found either above or below the crack. This zone consisted of small, blocky chunks of clay separated by fractures and varying in thickness from 1/8 inch to 2 inches. An exploratory trench was cut in the downstream face of the key, a distance of about 8 feet (Photo 12). The crack was visible throughout the trench, but the voids and width of the fragmented zone decreased gradually downstream while the percentage of bearing surface increased.

2.6 Preliminary observations were made in the downstream shear key on 21 September 1950 after 75 feet of the east section of the key had been excavated. A roughly horizontal crack had developed along the exposed section of the upstream face. The maximum overhang of the upper material averaged about 0.03 feet. After excavation of the key had been completed, two additional cracks were found: one extending from 35 feet west to 32 feet east; and the other from 25 feet east to 130 feet east, as shown on Plate 52. The apparent movement of the upper section into the key produced an overhang ranging from 0.02 to 0.10 feet along the western crack and from 0.02 to 0.17 feet along the eastern crack. Two trenches, shown on Plate No. 52, were dug into the upstream face of this key for a distance of 6 feet. The cracks were observed for the full depth of these trenches, but the size of voids and thicknesses of soft swelled material decreased in the direction away from the key with an accompanying increase in percent of bearing surface.

2.7 After preliminary observations of the crack in the upstream key had been completed, a decision was made to drill a number of large diameter auger borings for in-place inspection between the upstream and downstream keys to investigate the extensiveness of the crack under the powerhouse structure foundation. Six borings were drilled at the locations shown on Plate 50. Logs of the materials encountered and positions of cracks and other minor structural features are plotted on Plate 53. Two generalized sections showing the extent and approximate size of the cracks are given on Plate 51.

2.8 The key excavation and auger borings revealed a number of old structural features not related to the recent movement. Eight small, high-angle normal faults were found with dips varying from 45° to 75°, as indicated on Plate 49. Strikes were generally north-south deviating from N 23° E to N 10° W. Associated with most of the faults plotted were several small auxiliary faults having generally steeper dips, often in opposite directions, and terminating at the main fault. Displacements were small throughout the

area and throw averaged 0.70 feet with a range from 0.26 to 1.40 feet. In two of the main faults, which intersected thin lignite stringers, several fragments of lignite had been dragged along the fault plane, but on the whole, these planes were clean, slickensided fractures with no gouge material present. The faults in the upstream shear key terminated upwards at the base of the fat clay lens along which the crack from the recent movement formed. In the downstream key, the faults continued up to the top of the key excavation, as shown on Plate 52. On the upstream face of the upstream key, several small folds were present in the laminated silty clay and underlying fat clay lens, as illustrated in Photo No. 13. These folds were very local, extending a distance of about 20 feet along the face and appeared to fade out almost completely on the downstream face of the key. The folds were also interpreted as old structures developed to relieve minor stresses during consolidation.

2.9 During the same time that the movement and resultant cracks in the powerhouse structure foundation were being studied, the Contractor established 14 control points located along the upstream and downstream edges of the excavation, as shown on Plate 50, to determine whether any movement was taking place in the upper part of the foundation. This precaution was especially advisable along the downstream edge where excavation above the key had cut a vertical wall through a thick silty sand bed. Water draining from the overlying 2B lignite into the sand had saturated it and caused some sections along the face to slough badly (Photo No. 14) and it was feared the whole face might be in danger of collapse. A maximum movement along this face of 0.2 to 0.3 feet into the excavation was recorded at seven control points. A stable condition was reached in about 10 days. No appreciable movement occurred thereafter.

2.10 A possible cause of the movement may have been the unloading of the foundation by the 40-foot structural excavation below the rough excavation grade. Prior to making the saw cuts and key excavations, the only direction of active relief was upward; however, completion of saw cuts 3, 4, 5 and 6, provided initial relief of the lateral stress. Maximum deformation could then be expected to occur within Zone A which is represented in the field by that section of the foundation which moved into the key excavations producing the crack and associated loose, fragmented zones (see Plate 51). The actual position of the crack was determined, for the most part, by existing zones of weakness created by the low cohesion between the silt laminae and underlying fat clay along the upstream key and western section of the downstream key and between the lignite stringer and silty clay along the eastern section. In order to minimize the magnitude and extent of such cracks in remaining foundation excavations, it was suggested that outer saw cuts 3 and 6 be made first and allowed to stand for about 3 days before making inner saw cuts 4 and 5, thus permitting the maximum strain to develop in the key material to be excavated rather than in the foundation itself.

2.11 **EXPERIMENTAL UNDERWATER FILL STAGE III EMBANKMENT.** Investigations were made concerning the use of chunky Fort Union Clay-Shale for use in the diversion dikes of the Garrison Dam closure section. The Fort Union clay

chunks were excavated by power shovel and transported to the underfill site by bottom-dump Euclids. The location of the test site is shown on Plate 54. The construction started at 7:00 a.m., 15 October 1950 and was completed 9:05 p.m. of the same day. The material was dumped in windrows parallel and adjacent to the river bank. All material was placed by dozing over the end of the fill. Attempts were made to key the fill level about 5 feet above the river surface during the fill operations. The section was constructed 100 feet into the river and was maintained at the same elevation as the river bank. The width varied from 100 feet at the bank to 60 feet at the riverside top of the slope. A total of 5,050 cubic yards of chunky Fort Union Clay Shale was placed in the fill. The fill was later removed to preclude the apparent rapid siltation in the riverbed near the bank which was feared would silt shut the Contractor's nearby water supply intake. River soundings at the end of the fill indicated a water depth of about 15 feet and a velocity of 6 to 8 feet per second. No excessive erosion of the fill was noted under these conditions. Two 42-inch diameter test pits were augered in the center of the fill and moisture and density tests were taken at approximately 1-1/2-foot increments. Standard proctor test and consolidation tests were run on several samples.

2.12 The chunky clay shale used in the underwater fill test was the same as used in the Garrison Dam embankment, except the average dry density of the embankment rolled fill was 100 pounds per cubic foot as compared to 80 pounds per cubic foot for the underwater dumped fill. However, it was concluded that the dump fill would consolidate from construction traffic and would have a higher in-place density than indicated in the test. It was further concluded that the chunky Fort Union would be suitable and practicable for use in the closure dike and no problems were expected from erosion or saturation.

2.13 **PUMPING TEST IN ALLUVIAL FOUNDATION.** During the summer of 1949, tests were conducted in the river alluvium at a location about 1,200 feet downstream from centerline of the dam and approximately 1,000 feet east of the east channel of the river (see Plate 55). This location was selected as being typical of the foundation condition and was on or near the proposed relief well line. Two tests were conducted at this location and were referred to as Test No. 2 conducted in a gravel horizon and Test No. 3 conducted in a sand horizon. A test referred to as Test No. 1 had been previously run in a lignite bed. See previous paragraph 1.2.5, Chapter 2. The purpose of the tests was to investigate the overall permeability for the design of the relief wells and to explore the effect of stratification within the pervious zone. Plate 56 shows a generalized section of the pervious embankment foundation.

2.14 A 15 hp, A-C Cook, 6-inch, three-stage, vertical turbine pump with rated capacity of 600 gallons per minute was used for the test. It was powered through belt drive by an industrial type internal combustion unit using a tachometer and throttle to control the speed (see Photos 15 and 16). Water from the pump was carried in a pipe for a distance of 500 feet from the test well and discharged into a weir box and then into a ditch (Photos 15

and 16). The quantity was measured by an H flow meter made by Meriam Company of Cleveland, Ohio and a precalibrated knife edge V-type weir. The weir outlet was sheltered to prevent discrepancies in the reading due to the wind. Both devices registered approximately the same flows, but weir readings were considered the more reliable. The well screens were set as shown on Plate 55. In Test No. 2, a 15-foot long, 7-1/2-inch OD spiral-wound, slot type well screen with 0.12 inch slots (45 slots per lineal foot) was used. In Test No. 3, three 10-foot well screen sections were coupled for a 30-foot long screen. The screen was 6-1/2-inch OD spiral-wound slot type with 0.012 inch slots (70 slots per lineal foot). The design of the piezometers and observation wells used in the tests are shown on Plate 57. For Test No. 2, the well was constructed by setting a 10-inch diameter casing at elevation 1649.5 (the bottom elevation of the well screen) using standard churn drill procedure. The well screen was set and the casing was pulled to the top of the well screen. The annular space between the screen and casing was sealed off with a rubber gasket. The well was surged for a total of 0.6 hour, using a home-made surge block, and pumped for a total of 2.7 hours. The well was drawn down 16.7 feet and allowed to recharge. Approximately 0.65 cubic feet of fine sand and silt was drawn into the screen. For Test No. 3, the screen used in Test No. 2 was removed and a 30-foot screen was set using the same procedures as for Test No. 2.

2.15 The pumping tests were conducted on partial penetrating wells using Theim`s method of computing permeabilities. The summary of the tests is shown on Plates 57 and 58. Using the average actual thickness of the gravel and sand, and the average gradient between 60 and 138 feet from the well on Line A and Line C, an equation for the quantity of flow was written with the two unknowns being the permeability for the gravel, and the permeability for the sand. These equations were solved simultaneously for Test No. 2 and Test No. 3 and gave permeabilities for the gravel as $2{,}970 \times 10^{-4}$ cm/sec and for the sand as 92×10^{-4} cm/sec or a ratio of permeability of gravel to sand of 32 to 1. A study of flow net and piezometric data also bears out the fact that the gravel layer is much more pervious than the sand. The respective gradients in the gravel are much smaller than in the sand, indicating a higher permeability in the gravel. The average overall permeability of the foundation was computed as follows:

$$K \text{ average} = \frac{K_g d_1 + K_s d_2}{d_1 + d_2}$$

where

K_g = permeability of gravel = $2{,}970 \times 10^{-4}$ cm/sec.

K_s = permeability of sand = 92×10^{-4} cm/sec.

d_1 = thickness of gravel = 20 feet

d_2 = thickness of sand = 46 feet

$$K \text{ average} = \frac{(2{,}970 \times 10^{-4})\ (20) + (92 \times 10^{-4})\ (46)}{20 + 46}$$

$$= \frac{(59{,}400 \times 10^{-4}) + (4{,}232 \times 10^{-4})}{66}$$

$$= 964 \times 10^{-4} \text{ cm/sec.}$$

2.16 The following conclusions were made from the pumping test results:

(1) The permeability of the gravel is around 30 times the permeability of the sand.

(2) The average permeability of 1,200 x 10^{-4} cm/sec. used in design of relief wells was satisfactory.

(3) Permeability of sand can be estimated from laboratory tests on undisturbed sand samples. However, estimating permeability from grain size of churn drill samples is very questionable.

(4) Better developed wells, in which larger and more varied drawdowns are practical, would improve the test results.

(5) One hundred percent penetrating wells would have eliminated the three dimensional effect, permitting the use of data from piezometers located closer to the well and would have made the analysis easier.

2.17 **INVESTIGATIONS OF SLOPE FAILURE, INTAKE CHANNEL PLUG.** On 25 May 1952, a failure occurred on the riverside slope of the inner plug of the intake channel at Station 110+00. For location of intake channel plug, see Plate 59. For views of the slide, see Photos 17 and 18. Prior to the large slide, smaller slides had occurred at the toe during excavations in 1950. The plug was used continuously as a haul road and access road. As all of the temporary plug slopes at the Garrison Dam had been designed with minimal safety factor to minimize the excavations of plugs during diversion, only limited explorations were available in the area of the slope and subsurface conditions could only be generalized. The cause of the slide was not definitely determined but the following conditions were assumed to have contributed to the failure:

2.17.1 Weak alluvial clay was present at the toe of the slope as indicated by initial slide in the spring of 1951. The slope may have been in a state of incipient failure and a slight change in conditions may have triggered the slide.

2.17.2 Water surface elevations changed considerably. The spring flood was above elevation 1690 for only 60 days between April 3 and 8, inclusive, so it is doubtful that this had any effect on the slope that failed. However, the water surface was above elevation 1685 for such a long time that it is possible that the slope was under a drawdown of 2.5 to 3.0 feet. This may have been enough to trigger the slide.

2.17.3 It is possible that groundwater flow from the north bank of the intake channel could have helped in creating a partial quick condition at the toe that failed.

Repair of the plug slide was completed on 26 May 1952 and, except for minor cracking, the plug served its purpose until removed by the required excavations.

2.18 **OUTLET CHANNEL SLOPE FAILURES.** On 25 September 1951, a limited length of the west slope of the outlet works failed between Station 196+00 and Station 197+50 during Stage III excavations. For location of referenced outlet works stationing, see Plate 60. Views of the slide are shown on Photos 19 and 20. At the time the slide occurred, a dragline was excavating a drainage ditch near the toe of the slope. It was estimated that the slide included approximately 10,000 cubic yards. It appeared that the slide occurred when a fine, silty water-bearing sand was excavated at the toe of the slope by the dragline. This caused rapid lowering of the water table, establishing excess pore pressures in the fine silty sand, which when vibrated by operation of the dragline, caused liquification of the sand and excess stress to the material above. On 22 October 1951, a second slide occurred enlarging the original slide to include the area from Station 197+50 to Station 200+30. This additional sliding involved 25,000 to 35,000 cubic yards. The initial slide had opened large cracks in the vicinity of Station 197+50 and these cracks had continued to progress southward after the first slide. It was noted that water had ponded to a depth of several feet in a swale above the slide between Stations 198+40 and 199+50. The seepage of this pond into the adjacent areas of the old slide could have reduced the strength of the material in the second slide, already highly stressed, and caused the second slide. No remedial treatment was made on the slides as the slopes would ultimately be subjected to considerable erosion after the channel was placed in operation. However, the drainage ditch at the toe of the slide was relocated and the top of the slide was unloaded to the extent the dragline could maneuver.

3. **GEOLOGIC MAPS OF FOUNDATIONS.** At the time the Garrison Dam was constructed, there were no regulations or requirements for the preparation of final foundation maps of the structural excavations. However, a limited number of construction foundation reports were prepared that documented the construction procedures and methods of foundation preparations along with some reference to geologic conditions. Also, a few geologic maps were prepared for specific areas concerning potential underseepage and related problems. These construction related reports and maps are discussed in the following paragraphs:

3.1 LIGNITE OUTCROPS IN SPILLWAY EXCAVATIONS. In order to delineate the outcrop areas of lignites considered to be potential seepage paths for reservoir water, an outcrop map was prepared in the spillway excavations by surface mapping and drill hole correlations. Previous pumping tests in the BB lignite bed (see previous paragraph 1.2.5, Chapter 2) had shown the lignite to have relatively high permeabilities and correspondingly could serve as a potential horizon for underseepage. Plate 61 shows the location of the lignite beds exposed in the spillway excavations and projected areas. The map was prepared during the Stage II excavations and was used to design underseepage controls.

3.2 EAST ABUTMENT AREA LIGNITE OUTCROPS. Because much of the expected leakage through lignite beds from the reservoir would be diverted eastward by the east abutment grout curtain and spillway crest structure, a lignite outcrop map was prepared for this area to determine potential areas of seepage (see Plate 62). Although the maps were prepared on outcrops of the 2U lignite only, the 1U and the 1-1/2U lignites also follow a similar outcrop pattern and could be considered as potential leakage paths. The major lignites below the 1U lignite were not considered as a leakage problem in the abutment area as their downstream outcrops are either mantled with terrace deposits or the east flood plain alluvials.

3.3 CONTOUR MAPS OF THE LL, 1U AND 2U LIGNITES. In order to develop the planar characteristics of the spillway lignites, contour maps were made on the top of the LL, 1U and 2U lignites (see Plates 63, 64 and 65). These maps show the undulating and folding of the lignite beds, as well as the faults that traverse the area. The faults were believed to be a result of differential compaction during periods of superincumbent sediment loading or ice loading in the valley during Pleistocene glaciation.

3.4 CONTOURS ON BOTTOM OF WEST TERRACE GRAVEL. The west terrace gravel underlies the west bank terrace and in some areas is very close to ground surface. The gravel is very pervious and was used as a source of pervious borrow in areas upstream from the embankment. The seepage design for this section of the embankment foundation consisted of a cut-off trench, where the terrace gravel was shallow, and a sheet piling cut-off, where the gravel was deep. The downstream toe drain and the powerhouse switchyard drains also dewater this gravel. In order to develop the planar characteristics of the deposits and to assist in the design of the drainage and cut-off features, a contour map was prepared of the bottom of the west terrace gravel. The results of this study are shown on Plate 66.

3.5 TEST TUNNEL MAPPING. Geologic mapping was performed in the test tunnel to obtain information concerning the integrity of the Fort Union clays and lignites that might affect the construction of the eight tunnels comprising the outlet works for Garrison Dam. Most observations were made from the mining jumbo at the tunnel face as soon as practical after blasting. Orientation of the joints was obtained with a Brunton compass with the specific locations made by reference to adjacent structural steel ribs. The results of the mapping are shown on Plate 67. The stratigraphic and

structural characteristics were much as anticipated with the joints being nearly vertical and intersecting at somewhat less than 90° on the tunnel face.

3.6 **POWERHOUSE WEST SLOPE LIGNITE OUTCROP MAP.** The west slope of the powerhouse area includes several lignites and silt horizons that produce seepage. The seepage is controlled by a drainage system that discharges into the stilling basin. In order to locate the specific seep areas and determine the elevation of the various contributing horizons, a geologic map was prepared after final slope excavations showing outcrops and seepage points (see Plate 68).

CHAPTER 3 - GEOLOGY

1. **PHYSIOGRAPHY.** The Garrison Dam and Reservoir are located in the Missouri Plateau section of the Great Plains physiographic province. This region is divided by the Missouri River which generally separates the east glaciated region from the west unglaciated region. The Pleistocene glacial period greatly influenced the course of the Missouri River which flows across the Missouri Plateau, giving rise to a very complex drainage pattern. Prior to the glacial period, the Missouri River apparently flowed north into the Hudson Bay region, but egression of the Pleistocene continental glaciation altered its course to its present southerly direction. The Missouri River flows in a wide valley deeply entrenched in the Fort Union bedrock. The flood plains vary in width from 2 to 4 miles with the river meandering from bluff to bluff. In past geologic time, the river has cut much deeper into the valley than its present level. At the damsite, borings indicate that the river is flowing over alluvial and glacial fill deposits up to 200 feet thick.

2. **TOPOGRAPHY.** The damsite is included between relatively high valley walls about 11,000 feet apart reflecting one of the narrowest sections along the course of the Missouri River. See Plate 1 for location of the site. The profile of the original ground surface along the dam axis from east to west includes: a left (east) abutment at about elevation 1950; an east flood plain about 2,000 feet wide at elevation 1695; a river section (including an island) 3,000 feet wide and 10 feet lower than the east flood plain; a 4,000-foot wide west river bank terrace that rises gradually away from the river from elevation 1710 to about elevation 1760; and a right (west) abutment rising to about elevation 1900. The right abutment area was considerably gullied in contrast to the smooth rise of the left abutment.

3. **GENERAL GEOLOGY OF THE AREA.** The general geology of the Garrison Dam area is characterized by compacted sediments and lignite beds of the Fort Union Group, Tertiary age. During the construction period at the Garrison Dam, the North Dakota Geological Survey classified the Fort Union as a group, not a formation. Included in the group classification, in ascending order, were the Ludlow Shale Member, the Cannonball Formation, and the Tongue River Member. The Tongue River comprises the bedrock for the damsite. However, as all references to the bedrock at the damsite during investigation and construction referred to the Fort Union without respect to group or formation status, the damsite bedrock herein and hereafter will be referred to as the Fort Union.

3.1 At the damsite, the Fort Union as a group, is considered to be about 1,400+ feet thick based on data from the Definite Project Report, Volume 1, Basis of Design, January 1946. The Fort Union provides all of the structure foundations for the spillway, outlet works and intake structure and has a wide areal extent, underlying western North Dakota, northwestern South Dakota and eastern Montana. East and north of the site, the upland area is mantled with glacial drift that averages about 100 feet thick, but attains thickness in excess of 400 feet. West of the river, the glacial deposits fill preglacial valleys but the Fort Union Group generally forms the uplands.

4. DESCRIPTION OF THE OVERBURDEN. The Fort Union abutments at the damsite were mantled with glacial till as much as 50± feet thick and the slopes were covered with thin deposits of slope wash. As referenced in previous paragraph 1 of this chapter, the damsite borings indicated that the floor of the valley had been scoured by past glacial action to depths approximately 200 feet below the present valley floor. See Plate 40 for the geologic section. The overburden deposits that now fill the valley and comprise the foundation for the embankment are very complex. From the toe of the left (east) abutment to near the center of the valley, the deposits consist of alluvial silt and sand with intermediate and basal gravels. From the valley center to the right (west) abutment, the deposits consist of clay, silt, sand, gravel and glacial till. These materials were deposited by the action of the glacial ice moving across the area. Discussions concerning the gradation, permeabilities, and other soil characteristics will be included in subsequent paragraphs.

5. BEDROCK STRATIGRAPHY. The Fort Union bedrock supports all of the structures at the project. The bedrock is comprised of a series of alternating beds of moderately to well-compacted, gray to brown, stiff to hard clay (clay-shale), moderately to well compacted silt and fine sand, and numerous lignite beds. The lignite is usually firm to hard in its weathered state but slakes rapidly when exposed to air. The beds range from thin partings to more than 15 feet thick and are jointed and fractured and are usually water bearing. Thin limestone and sandstone beds and/or concretions occur sporadically throughout the Fort Union. The concretions are often very large, as much as 15 feet in diameter and 5 feet thick. The individual beds of clay, silt, and fine sand comprise the major portion of the Fort Union Group and vary from thin partings to beds over 15 feet thick. Slickensides were quite common in the fat clays adjacent to lignite beds. Only minor faulting was noted at the damsite. No evidence of deep-seated faulting exists in the Garrison Dam area. The materials comprising the Fort Union apparently became consolidated by previous loads of as much as 1,000 feet of overburden, either from past eroded deposits or glacial ice.

6. BEDROCK STRUCTURE. The bedrock at the damsite is essentially flat lying with only a slight regional dip to the northwest (dip equals about 15 feet per mile). Small flexures and local irregularities are common in the lignite beds. See Plates 63 and 64 for contours on lignite. Some indications of joint patterns were noted but there did not appear to be a well-defined system. Joint measurements in the lignites excavated in the spillway showed two predominate sets striking N 35° to 50° E and N 30° to 50° W. In general, most joints appear to be in a conjugate set and dip near vertical. The faults which were observed in the excavations are of the gravity or normal type. The only faulting of any importance occurred in the spillway excavations. For locations and sections, see Plates 61 and 69. Based on boring data as shown on the geologic section, Fault SF-1 had a displacement of less than 20 feet within the spillway excavations but the throw increased to about 100 feet in the area outside of the spillway excavations and downstream of the dam axis. Fault SF-1 was at first thought to be hinged near its intersection with the spillway crest structure, but during curtain grouting along the spillway structure, it was found to continue beyond the structure and

served as a cutoff of the lignite beds at this location. Fault SF-2 had a relatively small displacement of 5 to 10 feet and was not investigated in detail. The fault was believed to dissipate rapidly with depth. Faulting at these locations is probably the result of differential compaction by past heavy loading from glacial ice in the valley during Pleistocene continental glaciation.

7. **BEDROCK WEATHERING.** The Fort Union clays, silts, and sands did not include an appreciable amount of weathering that would effect a reduction in their strength properties. Most of the weathering was confined to color changes from oxidation processes. Exceptions to this were the lignite beds which were firm to hard in their unweathered state, but when exposed to air and moisture loss, readily slaked and became loose and broken. Where concrete was to be placed on lignite surfaces, care was required to protect the surfaces against drying and freezing. See Chapter 4 - Excavation Procedures. The limestone concretions included in the Fort Union also weathered quite readily on prolonged exposure as referenced in previous paragraph 2.2, Chapter 2.

8. **GROUND WATER.** Most of the ground-water information at the project was obtained from water level observations in the bore holes and pumping tests as discussed under previous paragraphs 1.2.5, Chapter 2, "Pumping Test BB Lignite Hole 608" and paragraph 2.13, Chapter 2, "Pump Test in Alluvial Foundation." In general, the regional ground water in the Fort Union was tributary to the river flood plain and terraces and movement was controlled by water levels perched in lignite beds or by sand layers. The pumping tests in the lignite indicated a range of permeability from 820×10^{-4} cm/sec. to 1100×10^{-4} cm/sec., which compared favorably with the permeabilities of the alluvial sands in the Missouri River valley. Ground-water levels in the flood plain alluvials east of the river were tributary to the river and generally ranged from 12 to 15 feet below the natural ground level of the flood plain. Levels in the west terrace were also tributary to the river occurring at depths of 60 to 70 feet at the toe of the left abutment and sloping to the river level. The chemical quality of the ground water from the Fort Union is given in the table below:

TABLE 4

CHEMICAL ANALYSIS OF WATER FROM THE FORT UNION

Calcium Carbonate	$CaCO_3$	390.00 PPM
Magnesium Carbonate	$MgCO_3$	178.50
Silica	SiO_2	15.05
Iron Oxide	Fe_2O_3	.00
Manganous Oxide	MnO	.51
Sodium Carbonate	Na_2CO_3	225.00
Sodium Sulphate	Na_2SO_4	975.00
Sodium Chloride	NaCl	10.25
Total Solids		1794.31
Hydrogen Ion		7.4
Carbon Dioxide	CO_2	34.20
Alkalinity (tems $CaCO_3$)		815.00
Total Hardness (terms $CaCO_3$)		603.00
Compensated Hardness		741.00
Suspended Matter	Little flocculent organic	

9. **ENGINEERING CHARACTERISTICS OF THE OVERBURDEN AND FORT UNION.** The engineering property values of the overburden materials and Fort Union used in construction of the embankment and structure foundations is discussed in the recently published "Garrison Dam Embankment Criteria and Performance Report," October 1981. This report is available from the files of Omaha District F&M Branch, as well as other concerned Corps of Engineers offices. The design data as presented in this criteria and performance report are presented in part herein:

9.1 **VALLEY GLACIAL TILL AND ALLUVIAL CLAYS.** A total of 79 stress-controlled consolidated-undrained direct shear tests and 37 unconsolidated-undrained triaxial tests with pore pressure measurements were performed on undisturbed samples of alluvial clays and glacial till. Results of the testing are discussed in detail in "Analysis of Design, Excavation and Embankment," May 1949. Out of the total direct shear tests, 58 were performed on terrace clays and surface clays and silts and 21 were performed on glacial tills. Little difference was found between the strengths of the till and overlying clays for either the direct shear tests or the triaxial tests and consequently both types of materials were combined for a representative strength. Pore pressure measurements and resulting effective strengths of the triaxial tests, used in comparison with the direct shear tests, could not be relied on and were not used in the design considerations. Only the results of the direct shear tests were used for selection of a design value for representative types of materials. The design shear strength for the alluvial clays and glacial till was selected as a lower than average value from the data and was C = 0.30 t/sq. ft., tan ϕ = 0.36. This value governed, to a large extent, the selection of embankment slopes. A shear strength summary of the tests is shown on Plate 70. Ranges of parameters from the

direct shear tests are as follows: Dry density 77 to 106 lbs/cu. ft.; moisture content 11 to 34 percent; clay content 7 to 94 percent; cohesion 0.05 to 1.5 T/sq. ft.; and tan $\emptyset$ = 0.14 to 0.68.

9.2 VALLEY ALLUVIAL SANDS. Twenty-two constant volume triaxial tests and nine constant lateral stress triaxial tests were performed on samples of the valley alluvial sands. From the constant volume triaxial tests, a total stress ratio of 2.0 corresponding to tan $\emptyset$ = 0.35 was conservatively selected for design use. This value, however, was never used in any of the stability analyses since the critical sections involved foundation clays rather than foundation sands. Typical gradations of the alluvial sands are shown on Plate 71.

9.3 FORT UNION. The Fort Union was tested for moisture content, dry density, mechanical analysis, compressive strength, shearing strength, consolidation characteristics, and modulus of deformation. All tests were conducted on undisturbed samples from drill holes.

9.3.1 Moisture and Dry Density. The Fort Union varies between 95 and 115 pounds per cubic foot dry density and moisture content ranges from 16 to 24 percent. As a general rule, the in-situ Fort Union can be considered to have a dry density of about 104 pcf and a moisture content of 21 percent. Mechanical analyses tests indicated a general range of soil types from fat clays to fine sands; however, the predominant soil type is a lean clay. Liquid limits range from 19 to 108 and plasticity indices range from nonplastic to 70.

9.3.2 Compressive Strength. The unconfined compressive strength of the Fort Union results almost entirely from its cohesion, not from cementation of the constituent particles. Tests indicate that the compressive strength varies from 2 to over 20 tons per square foot with strength between 2 and 6 tsf occurring more frequently.

9.3.3 Undisturbed Shearing Strength. Shear testing of the undisturbed Fort Union was conducted by both consolidated-undrained direct shear tests and triaxial tests. Both strain and stress control tests were performed for the direct shear tests. Consolidating stresses were generally between 2 and 7 tsf. All direct shear testing performed for the Definite Project Report, 1946, was stress controlled. Later tests were strain controlled and reported in the "Analysis of Design for Excavation and Main Embankment, 1948." Approximately 69 stress-controlled tests and 21 strain-controlled tests were performed on the material. Strain-controlled tests were sheared at the rate of 0.02 inches per minute. Approximately 37 unconsolidated-undrained triaxial tests were also performed on the Fort Union material. Test results for both types of tests were erratic with a fairly wide range of strengths, but the triaxial tests showed greater variation than the direct shear tests.

9.3.4 The design strength of the Fort Union material was assumed as C = 0.70 tsf and tan $\emptyset$ = 0.36, $\emptyset$ = 20′, and represented a value from the direct shear test data which was somewhat less than the average of

the results for both the strain and the stress-controlled results. The adopted value also appeared to be a fair average for the triaxial test results. Results of the testing for undisturbed samples are shown on Plate 72. Typical gradations are shown on Plate 71.

9.3.5 **Remolded Shearing Strength.** Remolded testing of the Fort Union was performed on material compacted by modified AASHO compaction procedures. Twenty-six consolidated-undrained direct shear tests were performed under normal loads up to 8 tsf on samples having dry densities ranging from 99 to 119 pounds per cubic foot. The adopted shear strength C = 0.2 T/sq. ft., tan Ø = .45, was selected from the lower part of the ultimate strength data.

9.3.6 **Consolidation.** A total of 52 consolidation tests were performed on the Fort Union material and the results generally supported the assumption that the material had been previously consolidated to loads much higher than any that would be imposed by the dam or structures. Results of the consolidation tests and seven absorption-pressure tests indicated to the designers that the theory of consolidation would not be applicable to the Fort Union foundation in terms of time rate of consolidation or to degree of pore pressure response to load conditions.

9.3.7 **Modulus of Deformation.** Modulus of deformation of the Fort Union material was determined by results of consolidation and triaxial compression tests. The average modulus of deformation from the 52 consolidation tests was about 500 to 600 tsf. The range of modulus as determined by the slope of the tangent of the stress-strain curves of 124 triaxial compression tests was found to be about 338 to 14,400 tsf and as a whole was generally substantially above the values from consolidation testing. It was thought that the difference was due to the inability to develop lateral pressures equal to those in the triaxial compression tests. Consequently, only the triaxial compression tests were used for selection of the design parameters.

CHAPTER 4 - EXCAVATION PROCEDURES FOR COMPONENT PARTS

1. **METHODS AND CLASSES OF EXCAVATIONS.** The excavation at the Garrison Dam project was accomplished by conventional ripping and scraping method, as well as backhoe and dragline. Vertical cuts in the bedrock for keys, drains and other finished faces, were often made with a "Joy" mechanical shale saw (see Photos 21, 22, 23 and 24). Occasionally, blasting was required to remove large limestone or sandstone concretions and to break up massive clay beds. Blasting was used routinely to excavate the tunnels. Discussions concerning the tunneling procedures will be described in subsequent paragraphs.

1.1 There were six designated classes of excavation for the Garrison Dam project: 1) Stripping excavations, which were limited to the removal of objectionable foundation material by stripping; 2) Unclassified excavations, which included all excavation except stripping excavations, waste excavations, lignite salvage excavations, trench excavation, mattress excavations and other miscellaneous minor excavations; 3) Waste excavations, which included all excavations in the Intake Channel upstream from Station 58+00 and all excavations in the Outlet Channel below Station 202+25; 4) Salvage lignite excavation, which included all lignite beds considered salvageable; 5) Trench excavations, which included the cut-off trench, exploratory and toe drain trenches and other miscellaneous trenches; and 6) Mattress excavations, which consisted of the removal and disposal of all materials required to permit the placing of a lumber mattress on the river bottom to serve as a base on which the rock diversion dike could be constructed for diversion and closure of the embankment.

2. **EXCAVATION GRADES.** The final grades for the Garrison Dam excavations conformed to essentially the same grades as required in the contract drawings. Only minor problems were encountered, such as small overbreaks in the lignite beds and other minor sloughing or raveling in the blocky Fort Union.

3. **EXCAVATION SLOPES.** The major final slopes of the spillway excavation varied from the 1V on 2H to 1V on 3H side wall excavations in the approach channel and spillway, to the vertical cuts for keys and lateral drains which were cut with a mechanical shale saw. From the crest structure to Station 40+40, the spillway slab is at elevation 1802. From there to Station 58+06.43, the chute slopes at 0.75 percent and beyond to Station 58+06.93 it slopes to 20 percent. The major slopes for the Intake Structure excavations varied from 1V on 1H to 1V on 3H. The slopes for the powerhouse varied from 1V on 1H to 1V on 2-1/2H. Vertical cuts were made with a mechanical shale saw for keys and drains. See Plates 73 through 79 for the required excavations.

4. **DEWATERING PROVISIONS.** Dewatering of excavation was routine and generally accomplished by sump and pump methods. A special dewatering provision was specified in the spillway stilling basin contract that required that the stilling basin be dewatered and maintained in a dry condition until the end of the contract. In addition, it was required that in order to prevent heave of the foundation as load was removed by excavation, the Contractor would depressurize the 2L lignite, located between elevations 1557 and 1581, and

the X lignite that was thought to lie in the southeast corner of the stilling basin between elevation 1584 and 1596. It was also required that water pressure in these beds be reduced during the period of structural excavations and subsequent wall and slab construction. After the permanent relief wells were installed, and the load added by construction exceeded the load removed during structural excavations, the depressurizing could be discontinued. The Contractor elected to install the permanent relief wells early in the contract and used them to depressurize the lignite beds.

4.1 Special dewatering practices were also required for the embankment foundation in preparation of the embankment closure. The area for this treatment was located between the upstream cofferdam and the sheet piling dike. The area was completely dewatered by open pumping and by a well point system. This was necessary so that the embankment materials could be placed in-the-dry. The contract also required that the foundation area between the sheet piling dike and the downstream cofferdam be dewatered by the same method, down to elevation 1670. The well points were needed in this area to control the flow of ground water into the dewatered area and reduce the possibility of boils developing in the unwatered river bottom. See Plate 80 for location of dewatering.

5. OVERBURDEN EXCAVATION. Following the grubbing and removal of roots from the embankment area, the contract required that all unsuitable material to include topsoil, vegetation, organic silt, swamp material and rubbish, as well as any other objectionable materials below the ground surface be stripped. The stripping included the embankment area and 10 feet contiguous thereto. It did not include the areas of the upstream impervious blanket. See Plate 81 for embankment layout. In addition to the stripping for the embankment, it was also required that the Contractor dewater and strip any sloughs encountered during the stripping operations. Conventional scrapers and draglines were used for this purpose. Preparation of the stripped area for embankment placement was accomplished by breaking down stump holes or other similar depressions and backfilling with appropriate material and properly rolling or tamping with a power tamper. The foundation was then scarified or otherwise loosened to a depth of approximately 6 inches, moistened if required, and compacted by 10 passes of the contract-specified roller. In the case of the foundation on the left (east) abutment slope, an inspection trench was excavated in the slope of the abutment to inspect the in-place condition of the overburden and top of the Fort Union bedrock. See Plate 84 for location of inspection trench.

6. ROCK EXCAVATION. As mentioned in preceding paragraph 1 of this chapter, the bedrock excavations at the Garrison Dam project were accomplished with conventional excavating equipment. It was required that all bedrock faces against which concrete was to be placed would be undisturbed, clean and damp, free from frost or frozen material, mud, standing water, or dried porous earth. All concrete was required to be placed as soon as practicable after excavation and approval of the foundation. During freezing weather, the Contractor was required to leave a protective layer of material above the foundation grade or line at least 5 feet thick. This protective layer could

not be removed for any foundation pour until just prior to concrete placement. No other special treatment was required to protect the Fort Union. The exposed lignite beds raveled slightly and because of their jointing and fracturing were subject to minor overbreaks. However, as most lignite beds were ground water carriers, they remained saturated on exposure and they did not readily break down and slake, as lignite does when mined and air dried.

6.1 **LIGNITE EXCAVATIONS AND STOCKPILING.** The general criteria for salvaging lignite beds encountered in the project excavations was to consider any bed 3 feet or more in thickness as a salvageable bed. The upper 6 inches and the lower 6 inches of each bed was wasted to assure a fairly clean lignite recovery. Euclid loaders were generally employed to strip the materials overlying the lignite beds and power shovels were used to excavate the lignite. See Photo 25 for lignite salvage in the outlet channel. For a geologic section showing the position of the lignite beds at the damsite, see Plate 40. In the thicker beds, such as the LL lignite, a ripper was used to facilitate the power shovels. Haulage from the excavation to stockpiles was accomplished with 30-ton Euclids. In order to determine the stockpiling characteristics of the Garrison Dam lignite, two small pilot piles were constructed in the area south of the present power plant. These two piles were designated as Stockpile 1 and Stockpile 2. See Plate 81 for location of stockpiles. Stockpile 1 consisted of approximately 10,000 cubic yards with basal dimensions of about 150 to 250 feet, 12 feet high and 1V on 4H side slopes. The lignite was placed in 3-foot lifts and received only traffic compaction. Stockpile 2 was of approximately 13,000 cubic yards with basal dimensions of 135 by 185 feet, 28 feet high, and had side slopes of 1V on 1-1/2H. This was much steeper than for the first stockpile and also different in respect to the placement in 2-foot lifts. Another experiment was the placement of an 11-foot thick impervious blanket on the side wall; however, this had some construction disadvantages and was removed. During the period from January 1948 to October 1948, the two piles were closely observed for spontaneous combustion and the Bureau of Mines made temperature and gas analysis surveys. As a result of the apparent favorable condition of these two pilot piles, Stockpile 3 was constructed. Stockpile 3 was built in three or four segments. During construction, a fire broke out in the northwest corner and required considerable effort to extinguish. As a result of this condition, a meeting was called with the Bureau of Mines and it was decided that the piles were receiving insufficient compaction. To assure a safe condition for the future piles, the specifications were changed so that each lift would be limited to 1 foot and would be compacted by four passes of a chopper or a sheep's-foot roller or a combination of both and the side slopes would be given additional compaction. With the adoption of this procedure, approximately 2,360,000 tons of lignite was successfully stockpiled. These piles have been reasonably stable and remain in a safe condition to this date. See Table 5 for a tabulation of data concerning the Garrison Dam lignite.

TABLE 5

GARRISON DAM LIGNITE DATA

1. **Approximate Tonnage for Each Stockpile**

Stockpile 1, 2 and 3 (combined)	281,000 Tons
Stockpile 4	208,500 Tons
Stockpile 5	774,000 Tons
Stockpile 6	476,500 Tons
Stockpile 7	287,000 Tons
Stockpile 8	333,000 Tons
Total	2,360,000 Tons
Reserved for Riverdale Heating Plant	360,000 Tons
Available for Disposal	2,000,000 Tons

2. **Fuel Analysis of Pile Depth Samples from Stockpile 5**

Date	1954		1955		1956		1957	
Basis	As rec'd	M.a.f.	As rec'd	M.a.f.	As rec'd	M.a.f.	As rec'd	M.a.f.
Proximate, percent								
Moisture	41.0		38.5		39.6		39.3	
Volatile matter	25.0	48.3	25.6	47.2	25.1	47.5	25.3	47.7
Fixed carbon	26.8	51.7	28.5	52.8	27.8	52.5	27.7	52.3
Ash	7.2		7.4		7.5		7.7	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Ultimate, percent								
Hydrogen	7.1	4.9	6.8	4.7	7.0	4.9	7.0	4.9
Carbon	36.9	71.2	38.3	70.8	37.2	70.3	37.4	70.6
Nitrogen	.6	1.2	.6	1.2	.6	1.2	.6	1.2
Oxygen	47.8	21.9	46.5	22.5	47.3	22.8	46.9	22.5
Sulfur	.4	.8	.4	.8	.4	.8	.4	.8
Ash	7.2		7.4		7.5		7.7	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
B.t.u./lb.	6,190	11,950	6,430	11,880	6,289	11,910	6,274	11,840

3. **Size Analysis of Stockpile 5**

Size Interval, Inches	Bench* 1	2	3	4	5	6	7	8	Aver-age, Per-cent	Cumula-tive Per-cent
Plus 4	8.2	15.0	8.4	7.7	10.4	10.0	7.6	6.7	9.0	9.0
4 x 2	17.8	17.4	15.2	14.3	13.8	12.7	14.7	11.7	14.3	23.3
2 x 1¬	8.2	8.8	9.5	8.3	8.7	8.6	8.9	7.9	8.6	31.9
1¬ x 1	6.8	6.8	6.5	6.6	6.3	6.4	7.1	6.1	6.5	38.4
1 x 3/4	7.4	7.3	8.2	7.3	7.5	7.4	7.9	8.3	7.7	46.1
3/4 x ¬	9.2	8.1	8.8	9.8	8.8	8.9	9.5	9.1	9.9	55.2
¬ x 3/8	7.3	5.6	6.6	6.8	6.4	7.1	7.0	6.9	6.7	61.9
3/8 x {	10.7	8.8	9.1	10.5	8.7	11.8	11.5	11.4	10.4	72.3
{ x 4-mesh	6.5	5.0	6.6	7.0	6.5	9.1	8.1	9.5	7.5	79.8
4 x 8-mesh	12.4	11.7	13.4	14.5	13.7	15.4	14.2	18.9	14.7	94.5
Minus 8-mesh	5.5	5.5	7.7	7.3	9.2	2.6	3.5	3.5	5.5	
	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

*Samples taken from eight vertical cut benches on south side slope of Stockpile 5.

6.2 **USE OF EXCAVATED MATERIALS.** Most of the materials required for the construction of the embankment were derived from the required excavations of the Fort Union. See Plate 85 for the earthwork schedule prepared during the design period. Plates 82 and 83 show the typical embankment section and zoning of specific materials. Only about 4 percent of the embankment materials were derived from separate borrow areas. Four types of material were used in the embankment section: 1) Impervious material, which was derived mainly from the Fort Union clays and ranged from lean (CL) to fat (CH) clays. The more plastic clays were placed in the upstream part of the impervious zone; 2) Random materials, which were derived from Fort Union lean and sandy (CL) clays and silts (ML); 3) Semi-pervious materials, which were derived from the sands and silty sands of the Fort Union, and silty gravels from the flood plain alluvials; and 4) Pervious sand and gravel, derived from Borrow Area "A" (adjacent to intake channel) and excavations in the intake channel.

6.3 Lignite fragments which became incorporated in the excavations of the Fort Union silts, sands and clays were allowed in the impervious and random fill section of the embankment if not more than 9 inches maximum dimension and were allowed occasionally in the pervious zone if not more than 12 inches maximum dimension. The amount of lignite allowed in the impervious and random zone could not exceed 20 percent by volume.

6.4 The Fort Union comprised most of the impervious and random fill used in the embankment and when excavated was often chunky and blocky. Because of this condition, it was necessary to break the chunks down prior to rolling. This was generally accomplished with a tamping roller. The moisture content was maintained on the high side of optimum to allow the chunks to knit together during the rolling.

7. **ROCK SALVAGED AND STOCKPILED.** Hard rock fragments from the excavation of the Fort Union limestone and sandstone beds, and limestone and sandstone concretions, as well as glacial boulders from the overburden, were required to be stockpiled for use in the rock dike. The rock dike was constructed during the embankment closure procedures. Approximately 50,000 cubic yards of rock were required prior to starting diversion of the river. See Photo 10 for views of typical rock stockpiles. The criteria for the rock to be used for this purpose required only that the deleterious material, i.e., lignite, rock fines, and soft rock in the stockpiles did not exceed that which would fill the voids between the larger rocks.

8. **SAFETY PRECAUTIONS.** Because of the good stability of the open cuts in the Fort Union, and in consideration that the deep cuts such as the spillway approach channel were designed to include berms, very little support was required in the Fort Union excavations. Some support was required in the tunnel portal areas which consisted of shoring and cribbing where fallout and overbreak occurred in the thick lignite beds (see Photos 26, 27 and 28). Continuous steel rib support was required in the tunnel excavation and will be described in subsequent paragraphs. Accident prevention and safety under the contracts required that the Contractor conform with safety requirements as outlined in the handbook "Safety Requirement" approved by the Chief of Engineers and as may be amended; and in accordance with safe practices set forth in the "Manual of Accident Prevention in Construction" published by the Associated General Contractors of America, the publications of the National Safety Council, and with the safety and sanitation laws and regulations of the State of North Dakota.

CHAPTER 5 - SPECIAL FOUNDATION FEATURES

1. **CUT-OFF TRENCH.** A cut-off trench filled with impervious material and a sheet steel pile cut-off was constructed to control underseepage through the alluvials comprising the embankment foundation. The cut-off trench was excavated through the shallow pervious strata and, for economic reasons, sheet piling was used for a cut-off through the deep pervious strata. The sheet piling will be discussed in the paragraph that follows. The cut-off trench was constructed on an alignment as shown on Plate 48. The slopes were excavated 1V on 1H for depths up to 25 feet and 1V on 2H for depths over 25 feet. The bottom width varied from 10 feet in the higher elevations to 20 feet in the valley excavations. A 10-foot overlap was required where the sheet pile cut-off and the cut-off trench interfaced. See Photo 29 for views of the cut-off trench construction.

2. **SHEET PILING CUT-OFF.** The sheet piling cut-off was constructed along the alignment as shown on Plate 48. See previous paragraph 1.2.10, Chapter 2, for a discussion of the test driving of steel sheet piling. The Contractor was allowed to drive the piling with either a double or single acting hammer using a protective pile cap. The use of a drop hammer was not permitted. Water jetting was allowed except for the last 5 feet of penetration. A tolerance of not more than 6 inches, plus or minus from the grade line was permitted. Provisions were made in the contract for pulling certain piles for inspection. See Photo 30 for view of the sheet pile construction.

3. **DETERMINATION OF PILING LENGTH.** Because of the probability that the impervious material in which the piling would be set could not be readily detected while driving, it was considered necessary prior to construction to establish the length of each piling for each respective location. These lengths and other data are shown on Plate 86. The predetermined pile lengths for the contract were established on the premise that the piles would penetrate 8 to 10 feet into alluvial clay and till but would penetrate only 3 to 5 feet into the harder Fort Union bedrock. These depths were determined from the investigational borings along the alignment.

4. **SHEET PILING SPECIFICATIONS.** The specifications required that the sheet piling be new Carnegie-Illinois Steel Company's M-112, Bethlehem Steel Company's SP-4, or equal, and weigh approximately 23 pounds per square foot of wall. Interlocking joints were required to have a direct tension of not less than 12,000 pounds per inch of interlock using joint length specimens of approximately 3 inches. The piles were required to be rolled steel conforming to Fed. Spec. QQ-S-741, Type II, Grade B, Class 1.

5. **UPSTREAM IMPERVIOUS BLANKET.** An impervious blanket was constructed upstream of the embankment to increase the seepage path and thereby reduce the amount of underseepage beneath the dam. The blanket was 1,250 feet wide and extended from embankment Station 42+00 to the east abutment (see Photo 40). At the east abutment, the blanket integrated with a smaller impervious blanket that covered exposed lignite beds at the approach to the spillway channel. The blanket was constructed of impervious Fort Union materials and

varied from 14 to 24 feet thick at its contact with the embankment, and from 5 to 14 feet thick at its upstream extremities. The lower 5 feet of the fill consisted of rolled impervious material and the remaining fill was placed in 12-inch layers and compacted by the construction equipment traffic.

CHAPTER 6 - TUNNELS

1. **TEST TUNNEL.** Based on recommendations at the January 1947 meeting of the Board of Consultants, which changed the outlet works design from a cut and cover scheme to tunnels, it was decided that a test tunnel was necessary to investigate the engineering and tunneling characteristics of the Fort Union. At the tunneling site, the Fort Union consists of fat clay, silty clay, silt, and thick lignite beds. See Plates 87 and 88 for the test tunnel geology. The test tunnel was excavated 36 feet in diameter as a portion of Tunnel No. 4. At the time the test tunnel was started, the Stage I excavations in the powerhouse area were sufficient to allow access to the Tunnel 4 portal area (see Photo 31). For speed in getting the test tunnel built, as well as to have the test section located beyond the zone of influence of load released by the powerhouse excavations, an access tunnel was constructed from the portal area into the abutment to the location of the section for the test tunnel.

1.1 **ACCESS TUNNEL.** The access tunnel was 9 feet by 11 feet, 213 feet long, and was constructed during the period from 29 March to 3 May 1948. The excavations were accomplished by blasting and hand mucking. Approximately 2 feet of lignite was left in the crown to provide a roof. Installation of timber support was allowed to lag behind the heading up to a maximum of about 60 feet. See Plate 89 for the general layout and details of the access tunnel.

1.2 **TEST TUNNEL.** After completion of the access tunnel, a vertical raise about 6 feet by 7 feet was excavated to the roof of the test tunnel and then widened out to full bore width. The excavation then progressed down to the invert creating a slot of full bore width and 6 to 8 feet long permitting erection of the first two ribs of the test tunnel. This enlargement to full bore from the access tunnel was then extended to Rib No. 9 (see Plate 89) to provide a length of about 20 feet for erecting the jumbo. See Photos 32 and 33 for views of the jumbo. After the jumbo was erected, full face mining was carried on for the balance of the tunnel.

1.3 **MINING METHODS.** Full face mining was a cyclic operation which consisted of drilling the entire face to a depth of about 6 feet, loading and shooting, placing crown bars, mucking out, erecting two ribs and then starting another cycle. Excavation and rib erection in the test tunnel, except for the south termination, was completed on 3 November 1948. The south termination, which was constructed intermittently during concreting operations, was completed on 22 January 1949. The concrete section of the tunnel was constructed in six longitudinal pours and three vertical lifts in the period 14 November 1948 to 17 January 1949. Blaw-Knox steel forms supported by the jumbo were used for the side pours up to the springline and the crown pour. The invert was poured without forms and screeded to grade. Concrete was mixed outside the tunnel and pumped into the forms. See Plate 90 for concrete sections and steel.

1.4 In contrast to what had been expected prior to construction, no significant amount of squeezing or swelling of the Fort Union occurred and the bedrock stood very well in the walls and heading. Because of the near horizontal bedding plane and a well pronounced joint pattern (see Plate 67), the blasted materials from the headings came down in block-like fragments up to a cubic yard or more in size. The blasting proved to be a much more expedient way of excavating the Fort Union than pneumatic clay spades which were effective in softer clays. Overbreak occurred in the crown up to 3 feet, but averaged about 0.7 feet for the entire tunnel. Exposure of the 1A and BB lignite beds to air, especially during cold weather when heated air was blown into the tunnel, caused some slaking of the lignite and also some raveling of the Fort Union clays. However, the effect of the slaking was not significant. Small amounts of seepage were encountered in the BB lignite exposed in the access tunnel and from the 1A lignite in the test tunnel. The seepage from the 1A lignite was controlled with some success by drilling and grouting from the heading. All points of seepage in the lignites were eventually grouted successfully from several points in the tunnel. For location of seepage points, see Plate 67. Construction of the test tunnel in general was far better than had been expected and showed that the Fort Union was safe material to tunnel.

1.5 **INSTRUMENTATION.** The instrumentation of the test tunnel was very comprehensive and beyond the scope of this report. However, the detailed results of the testing program have been documented in Garrison District reports, "Report of Test Tunnel," Part 1, Volume 1 and 2, August 1949 and Part II, Volume 1 and 2, July 1953. The following are excerpts from the above mentioned report that briefly describe the types of test measurements:

1.5.1 Determination of stress by means of strain measurements of the tunnel ribs with Whittemore strain gages.

1.5.2 Changes in shape of the tunnel ribs and concrete lining with precise extensometer tape reading to 0.001 inch.

1.5.3 Determination of earth movements and pore water pressures around the bore of the test tunnel.

1.5.4 Electrical instruments for measuring earth pressures on the slotted concrete section 4-D of the test tunnel and resulting strains in the concrete, reinforcing steel, and steel ribs.

1.5.5 Miscellaneous observations, including slippage at rib joints, compression of crusher blocks, soil sampling, etc.

1.6 The arrangement of the test tunnel portion of Tunnel No. 4 is shown on Plate 89. In general, the test tunnel was divided into five sections, described briefly below:

1.6.1 Section 4-A, a fixed rib section, was designed to allow yield of the Fort Union through open spaces between lags arranged in such a

manner as to form approximate squares bounded by the ribs and lagging. The section was divided into three sub-sections with ribs spaced 2, 3 and 4 feet on centers.

1.6.2 Section 4-B_1, a yielding rib section, was originally constructed with redwood crusher blocks, 2-1/2 inches thick, inserted at each joint between the rib segments to allow about 3 inches of radial yield by diameter shortening. Prior to mining Tunnel No. 5, it was feared that the yield necessary before the crusher blocks had crushed sufficiently to take additional load might cause an excessive load onto the soil pillar between the test tunnel and Tunnel No. 5. To prevent this possibility, this section of the test tunnel was lined with concrete.

1.6.3 Section 4-B_2, a yielding rib section, had redwood crusher blocks 1 inch thick inserted at each joint between rib segments which allowed about 1 inch radial yield. Since the crusher blocks in this section had crushed a larger percentage of their thickness and since the amount of additional crushing necessary to develop the full strength of the ribs was small, less concern was felt for this section and it was allowed to remain without lining as Tunnel No. 5 was mined past the test tunnel.

1.6.4 Section 4-C was designed as a fixed rib section, similar to section 4-A. It included two sub-sections, 4-C_1, expected to be the major measuring station most comparable to the main tunnels, and 4-C_2, a transition into the concrete lined section 4-D.

1.6.5 Section 4-D was the concrete lined section of the test tunnel containing four slots (one each at crown, invert and springlines) which were spanned with heavy 30 WF 210 steel beams. Measurement of stress in the steel spanning the slots served to weight the horizontal and vertical reactions. Section 4-D was divided into four sub-sections, 4-D_1, a transition; 4-D_2, the main measuring section; 4-D_3, a transition; and 4-D_4, the terminal section which was not slotted.

1.6.6 Prior to mining the test tunnel, a ventilation shaft was sunk from the ground surface and lined with telescoping steel casing to measure ground movement.

1.7 In general, the magnitude of the stresses and strains as measured in the steel support, concrete lining and Fort Union were much less than had been expected. The actual load as determined by the instrumentation was only a fraction of the designed load, which was 100 percent of the overburden loads. However, it was recognized that the relative accuracy of the observations were also less than had been expected, but even with the measurement errors, definite trends were apparent. For example, in all ribs the vertical diameter decreased and the horizontal diameter increased. In most cases, the vertical decrease was greater than the horizontal increase. In the fixed rib sections A and C the horizontal and vertical changes were about the same. In the crusher block sections B_1 and B_2, the vertical diameter changes were about four times the horizontal changes; and in the concrete section D, the difference between the vertical and horizontal

changes was larger than in Section A and C, but less than in Section B. Although large differences occurred in the stresses between adjacent ribs, as well as between observations on the same rib, the testing when considered as a whole showed the stresses in the steel ribs to be low, and especially low in Section B where yield was allowed by the crusher blocks. In general, the stresses increased the most rapidly immediately after the ribs were erected. The following is a summary of the vertical loads for which test tunnel section as determined from the test program:

TABLE 6

Summary of Loads

Section	Vertical Load - Percent of Overburden			
	November 1948	**February 1949**	**April 1949**	**July 1949**
A	14.7	16.7	17.9	15.4
B	4.6	6.2	3.1	3.8
C	7.8	9.2	3.8	7.9
D	8.1	16.6	18.2	37.3
Aug. A, C & D	10.2	14.2	13.3	20.2

1.8 **SQUEEZE TEST.** During the mining of the test tunnel it was noted that a thin layer of fat clay overlying the 1A lignite included an abundance of slickensides. As it was recognized that additional load would be imparted to the Fort Union pillars between the tunnels when the remaining tunnels were constructed, it was considered necessary that data be collected concerning the shear strength of the highly slickensided clay layer. To accomplish this, two in-place large scale squeeze tests were conducted to determine the shear strength of the slickensided clay layer. The test was conducted on the premise that the shearing strength of a thin clay layer could be obtained by increasing the compressive force on the clay until a plastic state was obtained. This was accomplished by confining the fat clay layer between the 1A lignite and a rigid plate. Plate 91 shows the squeeze test arrangement and location. Two large field tests were performed. For Test No. 1, shearing strength was 4.14 tons/sq. ft. and for Test No. 2, the shearing strength was 3.43 tons/sq. ft. The unconfined compression test which had been previously run on Fort Union fat clay without slickensides, but similar in respect to moisture, density and liquid and plastic limits showed shearing strengths of about 6 tons/sq. ft. Based on this comparison, it was postulated that the slickensides reduced the shearing strength of the fat clay to about 60 percent of the unslickensided clay.

2. MAIN TUNNELS.

2.1 **GENERAL.** There are eight tunnels through the west abutment that serve as a part of the outlet works. The tunnels are about 1,320 feet long, extending from the intake structure transition to the downstream portal walls located at the surge tank base. See Plate 87 for layout of tunnels. Tunnels No. 1 through No. 5 serve the five generating units and Tunnels No. 6 through

No. 8 are for river regulation and flood-control releases. Tunnels No. 1 through 5 (power tunnels) have an inside diameter of 29 feet, Tunnel No. 6 (regulation tunnel) has an inside diameter of 26 feet, and Tunnels No. 7 and No. 8 have an inside diameter of 22 feet. The tunnels were excavated in the Fort Union materials as described in the preceding discussions concerning he test tunnel. See Plates 87 and 88 for geology. Spacing between tunnels was about twice the diameter of the bore. Because of this relatively close tunnel spacing, which took into consideration a major savings of concrete and excavations for the upstream and downstream structures, simultaneous mining of adjacent tunnels was prohibited until a mined tunnel had been lined with concrete.

2.2 As the temporary slope of the downstream portals was cut steep (1V on 1H) and very high to minimize the length of the tunnels (see Photos 34, 35 and 36), portal excavations at the toe of the slope reduced the shear strength at the base of the slope and lowered the stability. To compensate for this situation, the portal structures were designed to maintain stability of the slope during construction and excavations, and construction of the portal was accomplished alternately to avoid reduction of stability at adjacent portals. The design of the downstream portals is shown on Plate 92. As will be noted, the collar slab section and barrel section are connected to each other by an articulating joint to permit considerable movement. The upstream or intake portal slope was much lower than the downstream portal slope, consequently the stability problem during construction was less critical (see Photo 38). These portals were designed similar to the downstream portals but lighter structures were used.

2.3 **TEMPORARY SUPPORT.** Circular steel ribs were selected to furnish temporary support for the Fort Union. Ribs were designed on the basis of a maximum working stress of 24,000 psi, saturated weight of overburden of 129 pounds per cubic foot and initially with a 3-foot spacing for all ribs to permit use of only one length spacer and lagging for the entire job. The original design for sizes of rib steel is given in the table below and was developed in November 1948 when only limited data from the test tunnel were available. Later designs were based on observations in the main tunnels which resulted in increasing the spacing to 4 feet and reducing the weight of the steel.

TABLE 7

	Load			
Dia. of Tun. (ft)	**Max. Overburden and Surcharge at Centerline of Dam (ft.)**	**Percent of Max. Overburden. Orig. Overburden Plus Dam (Assumed for Design of Temp. Support)**	**Steel Size @ 3' O.C. at Centerline of Dam Where Load a Max. (550 to 600' Tunnel Length)**	**Steel Size @ 3' O.C. Near Ends Where Load Less (300 to 350' Tunnel Length at Each End)**
29	145	50	10" WF 72	8" WF 48
26	150	40	10" WF 54	8" WF 40
22	160	40	10" WF 49	8" WF 35

The first purchase of ribs included both four and eight segments per rib, fabricated so that the four and eight segment pieces were interchangeable to permit flexibility for optional mining operations and to permit the use of both types to form one complete rib. Later, orders were limited to four segment ribs, as experience showed these to be best adapted for erection methods being used. To provide for ultimate action of the ribs as reinforcing in the concrete lining, the bolted segment joints were provided with splice plates which were welded to provide tensile strength in the joints of 18,000 psi across the entire section.

2.4 **CONCRETE LINING.** The concrete lining was designed to the following thicknesses:

Tunnel No.	Inside Diameter	Thickness of Lining
1 Thru 5	29'	3'-0"
6	26'	2'-9"
7 and 8	22'	2'-6"

Contact grouting through pipes at regular spacing and at points of high overbreak was required in the crown to fill any voids left in the crown after concreting.

2.5 **MINING PROCEDURES.** The Fort Union behaved as a soft rock during mining of the tunnels so that it was possible to mine an entire tunnel before starting to concrete. The mining was successfully carried forward by blasting and excavating for full height of the heading. The tunnels were advanced full face in pulls of 6 to 8 feet, depending on the spacing of the temporary support ribs.

2.5.1 **Jumbos.** Rail-mounted jumbos with platforms just below the springline kept the entire bottom portion of the tunnel clear and allowed mucking, steel erection, and drilling to be carried on simultaneously. Two jumbos were constructed for the 29-foot diameter power tunnels, and one each for the 22- and 26-foot regulating tunnels (see Photos 32 and 33). Hydraulic jacks mounted on traveling jacking cars were used to support the crown until steel was erected, and horizontal breast jacks bearing on double channels were used to support the vertical face and protect against local falls. The rear end of the jumbo was held down by kicker jacks pushing against the crown of a previously set rib.

2.5.2 **Drilling and Shooting.** During the time the steel ribs were being erected, the tunnel face was drilled starting at the top and working downward as the mucking progressed. By the time the invert was completely mucked and the invert segment of the steel rib was set, the entire face was drilled and ready for loading. Holes were drilled 6 or 8 feet into the heading by 1-1/2-inch coal augers with hand-held air drills. The drilling pattern varied slightly through the job to provide for variations in material. A typical cross-section and drilling and loading pattern is shown

on Plate 93 and consisted of 63 to 90 holes loaded with about 500 one-third pound sticks of Monobel dynamite. One primer stick of 40 percent dynamite was used for every three sticks of 20 percent dynamite. Shots were fired in standard delays numbered 0 to 11. The Fort Union drilled and shot very easily and required approximately 0.6 pound of powder per cubic yard of excavation. Mucking was accomplished with Conway muckers modified by means of a hopper and an air-operated double chute to permit loading alternate cars on a double rack.

2.5.3 **Erection of Ribs.** The arrangement of the jumbo permitted erection of steel ribs during mucking, the top segment being placed first. The crown segment was set in place and supported from the jumbo by means of hydraulic and screw jacks. The tunnel was supported from the ribs with wood blocking using a minimum of 16 blocking points per rib evenly spaced around the entire rib circumference. Open lagging was placed above the springline, except near the portals where solid lagging was used for the first 15 to 20 ribs. Where open lagging was used, the maximum distance allowed between lags was 3-1/2 feet center to center. Where additional support was needed, the lags were placed closer. A 2-inch by 4-inch wire mesh was specified in the top 180 degrees which caught local falls from the crown. The open lagging served to support this mesh (see Plate 93).

2.6 **CONCRETING.** Prior to placing the steel arch forms, the wire mesh placed in the crown of the tunnels for workmen protection was either removed or slit to allow additional scaling. If necessary, loose blocking and lagging was also removed. If it was not practical to remove the wire mesh, it was bent so as not to interfere with the concrete placement. The concrete was placed in 24-foot monoliths with the invert placed first followed by the arch and crown using Blaw-Knox Telescoping forms. The forms were stripped 24 hours after pouring and the concrete was water-cured for 14 days.

2.7 **GROUTING.** The contact grouting was accomplished in only the upper 120 degrees of the crown. The spacing and number of grout holes depended on the amount of overbreak and number of grout pipes placed during the concrete operations. A cement, sand, and fly ash mixture with an intrusion aid was used. All grouting commenced at the springline and progressed to the crown. The average grout volume used for tunnels was approximately 7-1/2 percent of the total volume of concrete placed in the tunnels above the springline.

2.8 **TUNNELING CHARACTERISTICS OF THE FORT UNION.** The tunneling characteristics of the Fort Union were essentially as encountered in the Test Tunnel. The Fort Union was very stable except for minor slaking and a few local fallouts. The blocky structure of the clay formed by the near horizontal bedding planes and vertical joints caused the greatest overbreak to occur on either side of the crown, often as an inverted stair as shown in the typical section of Plate 93. The average overbreak for all tunnels was only about 0.6 feet outside the back of the rib or "A" line, with 0.7 foot above and 0.5 foot below springline. The largest overbreak was in Tunnel 2 and extended 11 feet above the excavation line at the crown over an area of 12

feet by 15 feet. Very little water was encountered in mining the tunnels. However, a small amount of seepage was occasionally encountered in the 1A lignite near the springline and in limestone concretions occasionally found near the crown. Sump and pump methods were adequate to control the small amount of seepage from the lignites. In general, the tunneling characteristics of the Fort Union reflected stable mining conditions. See Photo 39 for a typical tunnel heading showing the 1A and BB lignite between Fort Union clay-shale.

CHAPTER 7 - FOUNDATION ANCHORS

1. **SPILLWAY ANCHORS.** The spillway chute and stilling basin pavement was designed with foundation anchors to resist uplift pressures. Two types of anchors were used in the spillway. Type A anchors were used from the crest structure to about Station 63+50 and again between Station 67+50 and Station 68+50. These anchors were set on 8-foot 4-inch centers and were fabricated from #11 steel bars hooked at the top and fitted with a welded plate at the bottom. See Plate 94 for design of the Type A anchor and anchor hole. Type B anchors were used from Station 63+50 to Station 67+50. These anchors were fabricated from #16 steel bars with a plate welded at both the top and the bottom. The anchors were spaced on 15-foot centers from Station 63+50 to Station 64+00 and from Station 67+15 to Station 67+50, and on 10-foot centers from Station 64+00 to Station 67+15. See Plate 95 for design of the Type B anchor and anchor hole. The bars were coated with a plastic bituminous cement to prevent corrosion. All anchor holes were drilled normal to the slope to decrease the shear on the anchors. Anchor pullout tests to determine the strength characteristics of various types of anchorage in the Fort Union were previously discussed in paragraph 1.2.11, Chapter 2.

1.1 The contract gave the Contractor the option of constructing the anchors prior to placing the pervious fill for the concrete slab or he could construct the anchors after placing the pervious fill if he conformed to the following requirements: 1) For all temporary or permanent anchors, casing was to be installed through the pervious fill and screened gravel in the slab areas. Drilling of anchor holes and belling was to be done without using water; 2) For Type B anchors adjacent to the drain, a length of steel casing was to be set prior to placing drain pipe and the anchor was to be subsequently constructed through this steel casing. The casing was to extend 2 feet below the drainage trench and remain permanently in place; 3) For all anchors adjacent to drains, the Contractor was to exercise extreme caution to prevent any impairing of the drain's effectiveness. If any damage to a drain occurred, he was to reconstruct the drain to the satisfaction of the Contracting Officer.

2. **DRILLING OF ANCHOR HOLES.** The Contractor was allowed to drill or auger the anchor holes. The tolerance in alignment was not to exceed 1/4 inch in 1 foot of depth. The shaft portion of the hole could be permanently or temporarily cased. If the belled portion of the anchor hole bottomed in hard strata, such as a concretion, or interval of unstable materials, the hole could be deepened as required to achieve a belled bottom. If a hole was allowed to remain open more than 24 hours prior to placement and concreting of the anchor, the anchor hole was required to be covered or plugged. If a hole was abandoned, a new hole location could not exceed a distance of more than 18 inches from the original hole.

3. **PLACEMENT OF ANCHORS.** Placement and concreting of the anchors required that the anchor holes be dry insofar as practicable, but if a dry condition could not be obtained, the concrete could be placed under water. If the

concrete was placed under water or if the anchor hole was uncased, the concrete was required to be placed by a tremie. In dry cased holes, the concrete could be dumped in at the top. The Type A anchor design did not allow the use of a coarse aggregate concrete. The Type B anchors allowed a coarse aggregate but not to exceed a maximum size of 1-1/2 inch. The exact proportions of Portland cement, fine aggregate and coarse aggregate was determined by the Contracting Officer. When directed, aluminum powder up to one-hundredth percent of the weight of the cement could be used to achieve a fast set.

CHAPTER 8 - CHARACTER OF FOUNDATION

1. **FOUNDATION SURFACES AND CONDITION OF ROCK**. The major final foundation surfaces for the structures at the Garrison Dam were founded on the Fort Union. The clay-shales, silts, sands and lignites that comprised the Fort Union proved to be stable foundation materials except for minor blocky fallouts of the clay in a few excavations, and some raveling and slaking of the lignite beds and fat clays when allowed to dry out. Very little overexcavating or backfill concrete was required on the final cuts and slopes. Most vertical cuts were accomplished with a shale saw which minimized any disturbances to the excavated face. Ground-water seepage through sandy horizons and the relatively pervious lignite beds was generally handled by conventional sump and pump methods.

1.1 **UPSTREAM PORTALS AND INTAKE STRUCTURE**. The upstream portals and intake structure are founded on alternating beds of moderately to well-compacted silt and fine sand, very stiff to hard clay (clay-shale) and lignite beds. See Photo 40 for preconstruction view of intake and portal area. See Photo 38 for portal construction. The clays predominated and often included concretionary zones and thin limestone beds. The lignite beds ranged from thin partings to beds over 8 feet thick. See Plates 41 and 96 for geologic sections through the portals and intake. No unusual or major foundation problems developed during the portal and intake structure foundation excavations. Minor sloughing of the clay and lignite occurred at areas which became saturated or were over-stressed by the blasting used to loosen the bedrock. An example of this type of overbreaking occurred in the Portal No. 4 excavations where clay interbedded between lignites became saturated by ground water and sloughed out, leaving a one- to two-foot overhang of the BB lignite bed. For the most part, the construction foundation reports indicated relatively good foundation condition for the intake structure and portals. A minor problem developed at the abutment pier of the intake structure service bridge during July 1951. After partial completion of the pier, movement took place in the foundation materials that resulted in adverse tilting of the pier. As the pier could not be salvaged, the abutment design was modified and the pier was reconstructed using steel H piling for pier support. The subsurface condition and the pile design and pile driving data for this problem area are shown on Plate 97. Views of excavations for bridge Piers 1 and 2 are shown on Photos 41 and 42. See Plate 77 for plan of excavation for the intake area. The completed intake structure is shown on Photo 57.

1.2 **DOWNSTREAM PORTAL, SURGE TANKS AND POWERHOUSE**. The foundations for the referenced features are founded on Fort Union strata varying from clay-shale to silty sand. The clay beds ranged from partings to 15 feet thick and were interbedded with lignite beds up to 8 feet thick. The clays were impervious but the lignites were well-jointed and controlled a large amount of ground water movement. A geologic profile and section through the powerhouse area is shown on Plate 41. Stage I excavations for the west powerhouse slope include drainage berms on the bottom of the 1A and 2A lignite beds in order that seepage from these lignites would not erode the

west powerhouse slope. See Plate 78 for plan of excavation for the powerhouse area. Because of the varying types of foundation materials and the possibility of differential settlements, the surge tank units were constructed as independent monoliths with contraction joints between units. Uplift and settlement of the powerhouse and surge tank structures will be discussed in a subsequent paragraph.

1.3 In general, the foundations for the portals, surge tanks, and powerhouse were of good quality and no serious foundation problems were encountered. Most final vertical cuts were made with a mechanical shale saw and in the downstream portal areas the thick lignite beds (1A and BB) were often line drilled, allowing for good final surfaces. See Photos 34 and 35 for views of portal excavations in the clays and lignites. See Photo 37 for surge tank, powerhouse and stilling basin area. Minor movements occurred in the powerhouse foundation during excavations for the upstream and downstream shear keys. The movement was first noted in saw cuts about 4 inches wide which were cut along the shear key alignment to facilitate shovel excavations. These cuts closed shut in some areas and cracks developed in portions of the key excavations. Some small, high angle, normal faults were also encountered in the key excavations. The faults had throws of 0.26 to 1.40 feet, but were not considered related to the referenced movements. The movements were considered a result of stress relief caused by unloading and did not affect the integrity of the powerhouse foundation. The powerhouse foundation movement has been previously discussed in paragraph 2.5, Chapter 2. See Photo 58 for completed powerhouse and surge tanks.

1.4 **TUNNELS**. The foundation conditions encountered in the tunnels have been previously discussed in Chapter 6, and will not be repeated under this section.

1.5 **SPILLWAY**.

1.5.1 **General**. The foundations for the spillway approach channel, crest structure, chute, stilling basin, and discharge channel were founded on Fort Union deposits varying from clay-shale to silty sands ranging from a few inches thick to 15 feet thick. Within these deposits were numerous layers, lenses and zones of limestone and sandstone concretions, generally less than 2 feet thick, and numerous lignite beds ranging from thin partings to more than 12 feet thick. See Plate 42 for a generalized geologic section along the spillway centerline. Boring layouts for the spillway are shown on Plates 2 and 29. The logs of borings are shown on Plates 30 through 36 and the plan of excavation is shown on Plates 73 through 79. Two well pronounced faults, SF-1 and SF-2, were detected by exploratory borings, but were not visually observed in the foundations. A description of these faults was given in previous paragraph 6, Chapter 3. Excavated slopes in the spillway were good and encountered only a minor amount of slope instabilities, such as sloughing and raveling of lignite beds in areas where the lignite was allowed to dry out, and minor raveling of air dried, fat clay exposures. Occasionally, small overbreaks occurred in the clay, which excavated very blocky. Vertical cuts for the rest structure foundations and vertical cuts for drains were accomplished with a mechanical shale saw which assured a competent finished face.

1.5.2 **Crest Structure**. The foundation for the spillway crest structure was excavated to a depth of approximately 33 feet with the key portion of the structure extending to 41 feet. See Plate 98 for the design of the spillway crest structure. Excavations for the foundation encountered the 2U, 1-1/2U and the 1U lignites interbedded in Fort Union clay-shale. The construction foundation reports for the spillway structures stated that no unusual foundation conditions were encountered. A small amount of ground water seepage occurred at the base of the 2U and 1-1/2U lignite beds on the upstream side of the excavation in an area extending from the centerline of the spillway to 100 feet west of the centerline. The seepage varied from 30 gpm in the spring to 3 gpm in the fall. The seepage was collected in a sump and pumped as required. The bottom and side slopes of the crest structure key were thoroughly cleaned of loose and wet materials prior to placing concrete. Other than this routine procedure, no special foundation treatment was required. See Photos 43 and 44 for views of the crest structure foundation.

1.5.3 **Chute and Drainage Trenches**. A review of the foundation reports prepared during construction of the spillway chute, walls, and drains did not reveal any significant foundation problems. When lignite beds cropped out in the finished chute foundation or bottom of the drains, the beds or seams of lignite were left in place unless the finished foundation surface was disturbed, in which case the disturbed portion was overexcavated to firm material. Plates 99 and 100 show the area in the chute where lignite was not removed from the foundation surface. In intervals where the drains were designed to intercept lignite beds for drainage purposes, the lignite walls of the drain trench were cleaned with wet burlap to open joints and fractures, thereby permitting free drainage into the drains. See Photos 45, 46, 47 and 48 for views of final slopes and cuts for chute floor, chute walls and drains.

1.5.4 The final foundation surface of the spillway chute was smoothed with a scraper and pervious backfill was placed over the entire area, including the previously backfilled drainage trenches. The final chute foundation was first covered with 6 inches of pervious material to permit trucks to traverse the area during wet weather. The lift was later built up to 24 inches, sprinkled and rolled, after which an additional 18 inches of pervious material was placed, watered and rolled. The entire area was then bladed and made ready to receive the concrete slab. The pervious material was obtained from Borrow Area B and consisted of clean bank-run sand and gravel.

1.5.5 **Stilling Basin**. The stilling basin was founded on essentially the same materials as the chute. The construction foundation reports do not indicate any unusual foundation conditions except a minor overbreak in the Fort Union clay-shale encountered in the basin excavations for the end sill. This interval required an interceptor drain to dewater the lignites (see Plate 101). A small fault or closely jointed interval was responsible for the overbreak in the Fort Union clay-shales (see Photo 49).

Other than this small overbreak area, which required backfill concrete, the foundation surfaces were sound and durable and needed only the removal of loose and wet materials prior to the placement of structural concrete. See Photo 50 for foundation of the sill before final cleanup. Photos 51, 52 and 53 show the stilling basin foundation and views of spillway under construction. Photo 58 shows the completed spillway.

1.6 **EMBANKMENT**. The foundation materials for the embankment consisted of alluvial silts, sands, clays and gravels, as well as thick deposits of glacial till. See Plate 40 for the geologic profile along the centerline of the dam. From about Station 20+00 to Station 62+00, the embankment is founded on thin alluvial deposits of silt and clay underlain by sand and gravel. A stratified, stiff, plastic alluvial clay which averages about 50 feet in thickness, underlies the sand and gravel to about Station 90+00 and is in turn underlain by a deposit of glacial till. The till is about 60 feet thick and is often intercalated with sand lenses. The till and the overlying plastic clay deposits were considered to be the weakest foundation materials for the embankment. The design of the embankment was controlled by the strength of these deposits. From about Station 80+00 to station 118+00 (east flood plain) the embankment is founded on an impervious natural blanket of alluvials varying from silt to clay. These deposits ranged from a few feet thick to more than 25 feet thick. Underlying the impervious blanket are thick deposits of silty sand and fine sand including some intermediate gravels. These valley sands also extend into the river interval of the embankment foundation between Station 65+00 to Station 80+00. The preparation of the ground surface on which the embankment was founded has been discussed previously in paragraph 5, Chapter 4. See Photo 54 for a view of the initial embankment construction during Stage I earthwork. Photo 60 shows the completed embankment.

CHAPTER 9 - FOUNDATION TREATMENT

1. GROUT CURTAIN.

1.1 **GENERAL.** The Garrison Dam grout curtain in the west abutment extends from the west end of the sheet piling to about 1,500 feet west of the tunnels. The grout curtain in the east abutment extends from the east end of the sheet piling eastward to a small fault designated Fault SF-3. For location, see Plates 48 and 104. The grout curtain in the west abutment was constructed upstream of the embankment centerline, beginning at a point where the underlying 2B lignite is abutted by glacial till in the valley fill. Grouting in the west abutment was carried to the bottom of the 2B lignite (see Plate 40 for location of 2B lignite) and was constructed in four sections. In addition, a short grout curtain was located immediately upstream from the powerhouse and around the east and west sides of the powerhouse. See Plate 102 for location of the west abutment treatment and Plate 103 for a typical grout section.

1.2 The grout curtain in the east abutment started 650 feet upstream from embankment centerline Station 113+00. This location overlaps the sheet pile cut-off about 500 feet and is at a point where the underlying 1L group of lignite beds abut the basal gravels in the valley fill. Grouting in the east abutment was carried to the bottom of the 1L group of lignites. See Plate 104 for a generalized geologic profile of the east abutment grout curtain.

1.3 The purpose of the grout curtain was to reduce or control seepage through the lignite beds by grouting the open fractures and joints. The lignite horizons in the east abutment in ascending order were the 1L group, LL, 1U and 2U. The individual beds varied from 1 to 12 feet in thickness and were located above elevation 1625. The LL lignite correlates with the BB lignite on the west abutment. Correlation of the west lignites with the east lignites was accomplished on this basis (see Plate 40). Curtain grouting through the tunnel area was done from inside the tunnels with radial holes spaced entirely around the circumference of each tunnel. See Plate 102 for location of tunnel curtain.

1.4 **GROUTING CRITERIA.** On the basis of early grouting experience at the damsite, the grout quantities injected in a specific lignite horizon were generally limited to not over 10 cubic feet of grout per 1 foot of lignite. If a seam took less than 7 feet of grout at maximum allowable pressure, it was considered tight. The allowable grouting pressures at the packer settings in the overburden materials could not exceed the weight of the overlying material, and the allowable grouting pressures at the packer settings in the Fort Union could not exceed the pressures as given in the OCE Engineering Manual Foundation Grouting (EM 1110-2-3503), dated August 1950, grout pressure curve for stratified rock. In no event were pressures in excess of 75 psi, as measured at the top of the hole, required. During the initial grouting of Section 1 in the east abutment, the grouting pressures were limited to 50 psi, irrespective of the depth of grouting, because of higher than expected grout takes. However, the reduced pressures did not

result in any significant reduction in grout quantities and the reduced pressure criteria also appeared to increase the time required to grout a specific horizon. This criteria was omitted from the remaining east abutment grouting.

1.5 **GROUTING METHODS**. The spacing for grout holes was on the basis of the "split-spacing" method. The primary holes were spaced on 40-foot centers with split spacing employed until the curtain proved tight. Grouting in the west abutment was generally completed at the 10 foot spacing. This proved adequate as evidenced by tight holes on 5 foot spacing. However, spacing in the east abutment was often carried to the 2-1/2 foot spacing before achieving a tight curtain.

1.6 Both "stage" and "stop" grouting were emloyed for the grout placement, but most grouting was done with the "stop" grouting method. In "stage" grouting, the grout hole was drilled to below the first horizon then washed and grouted with the process repeated for each underlying horizon. In "stop" grouting, the hole was drilled full depth and each horizon individually grouted from the bottom up using a packer at the top of each horizon.

1.7 **EQUIPMENT**. The equipment used for grouting was variable as both hired labor and contract grouting was employed at the project. The contract drilling required that the equipment conform to the following specifications:

1.7.1 **Drilling Equipment**. Holes could be drilled from the surface of the ground to the top of the highest horizon to be grouted by any type of drilling rig using any method of advancing the hole. Below the top of the highest horizon to be grouted the hole was to be drilled with a rotary type drill using circulating water, but any type of bit could be used which would drill a satisfactory hole having a minimum diameter of three (3) inches. When the drilling penetrated to below the top of the highest horizon to be grouted, the consistency of the drilling fluid was to be thinned and regulated to the satisfaction of the Contracting Officer to avoid premature stoppage in underlying horizons to be grouted.

1.7.2 **Grouting Equipment**. All equipment used for mixing and injecting grout furnished by the Contractor, was to be of a type and capacity approved by the Contracting Officer, and was to be maintained in first-class operating condition at all times. Each complete grouting unit was to consist of the following minimum equipment:

(1) Two specially equipped duplex, air driven, double-acting slush pumps, capable of operating at a maximum discharge pressure of 100 psi.

(2) A mechanical grout mixer.

(3) A mechanically agitated sump.

(4) A tank for auxiliary water supply to be used in pressure testing, flushing and washing operations.

(5) A suitable water meter graduated in cubic feet and tenths.

(6) Such valves, pressure gages, pressure hose, supply lines, and small tools as may be necessary to provide a continuous supply of grout and accurate pressure control.

(7) Expansion packers which could be seated and which would remain seated and prevent leakage at depths and pressures to be used in the work. (In grouting accomplished by the Government, a patented hydraulically expanded rubber cylinder type packer was successfully used and was available for inspection at the site of the work. This type of packer could be used without payment of royalty on this job.) The capacity of the above listed plant was to be not less than 40 g.p.m. when operating at a discharge pressure of 75 psi. The inside diameter of the pressure hose and grout supply line was to be not less than 1-1/2 inches.

1.8 **WASHING AND PRESSURE TESTING.** All grout holes were required to be thoroughly washed immediately upon completion of the hole and after completion of the grouting of each horizon. Each horizon that was to be grouted was pressure tested with clean water under continuous pressure, but not exceeding the maximum allowed by the Contracting Officer. Horizons in which the maximum pressures could not be reached were washed as long as there was an increase in the rate of flow, or a drop in pressure with the pump delivering capacity flow. Where open horizons were encountered and where no pressure could be built up, the holes were washed for a period of five minutes or as directed by the Contracting Officer.

1.9 **GROUT MIXES.** The grout for the west abutment consisted of a neat cement mix with water:cement ratios generally varying from 4.0 to 0.6 parts of water to 1 part cement. For economy reasons, the mixes for the east abutment grouting included fly ash as a supplement to the cement and later the fly ash was mixed or replaced with river silt. Much of the grouting in the east abutment was accomplished with a mixture of 1 part cement to 1 to 2 parts fly ash; or 1 part cement, 1 part fly ash and 2 parts river silt. Ordinarily, silt was not used in a grout mix in which the water:cement ratio exceeded 2 parts water to 1 part cement. For starting a horizon, or in a tight horizon, a neat cement was used with a water:cement ratio ranging from 3:1 to 6:1. Daily drilling and grouting records for the various referenced grout sections are on file at the Garrison Dam area office, but there are no compiled records available concerning the total quantities of grout used for a specific grout section. However, as an indicator of these quantities, approximately 23,500± sacks of cementing material (cement ± alfesil) were used in the first 1000± feet of the east abutment grout curtain. Grouting in the west started in July 1949 and continued intermittently through 1953. Grouting in the east abutment started in August 1951 and continued through 1955.

1.10 CHANGES IN GROUT CURTAIN DESIGN. During grouting of Section IIIB, east abutment spillway grout curtain, the grout hole drilling encountered fault SF-1 which, prior to the grout drilling, had been suspected, but not proven, to extend to the north and intersect the middle of the crest structure foundation (see Plate 63). As this possibility was considered during the preparation of the Section IIIB grout contract, a provision had been written into the contract omitting the grouting of the LL lignite and deeper lignites if the fault was found to intersect the grout curtain and offset the lignite beds. On this basis, the grout curtain east of the fault was not constructed below the 3X lignite. This resulted in a large saving in the construction cost of the east abutment grout curtain. Close observations of strategically located piezometers confirmed this leakage cut-off condition in the LL and lower lignites.

1.11 SECTIONS REQUIRING ADDITIONAL GROUTING. Section III of the west abutment grout curtain was recognized as being in an area of possible rebound due to foundation unloading. This grout section is about 500 feet long extending from near Tunnel No. 8 to the top of the intake structure slope excavation. The interval was grouted in 1949 (see Plate 102). Because of the possibility of rebound disrupting the grout curtain, approximately 400 feet of the curtain was regrouted in 1950 with a grout take of about 60 percent of the original take. In 1953, the remaining 100 feet were regrouted when the curtain was extended westward. This interval had a grout take of about 100 percent of the original take, attesting to considerable reopening of the original grouting by the rebounding.

2. DRAINAGE PROVISIONS. The drainage provisions to control underseepage beneath the dam and through the abutments consists of an elaborate system of horizontal and vertical foundation drains. This portion of the foundation report will be concerned primarily with the design and construction of the major, permanent foundation drainage provisions. The provisions for dewatering during construction were relatively minor and have been discussed in preceding paragraphs. See Plate 105 for location of the various drainage systems.

2.1 EMBANKMENT TOE DRAINS AND RELIEF WELLS AND EAST ABUTMENT DRAINS. Provisions for control of seepage through the embankment consisted of a toe drain installed in the pervious horizontal blanket (see Plate 105). Within the old river section, the toe drains also came in contact with the foundation sand and served to collect underseepage. Monitoring of the toe drain by piezometers has not indicated any evidence of through seepage in the embankment.

2.2 Provisions for the control of underseepage beneath the embankment was accomplished with an elaborate relief well system at the downstream toe of the embankment. See Plate 106 for the plan and profile of the relief wells. The system consists of 54 relief wells. Forty-seven of the wells penetrate intervals that include about 80 percent of the total thickness of the valley sands, and seven wells penetrate the east abutment LL and 1L lignite beds. In addition to the relief wells at the toe of the embankment, one well (Well No. 57) was constructed opposite the sheet piling at Station

35+00. This well is located in a manhole for the downstream toe drain and is installed in the west terrace gravel layer.

2.3 The relief wells discharge into a drainage ditch paved with bank-run gravel and rock spalls. The gravel and spalls act as a weighted filter in resisting upward seepage pressures that may occur midway between the wells. The well spacing and depth of penetration into the valley sands was adjusted to obtain a safety factor of about 1.5 against flotation in the bottom of the well channel. Plate 107 shows the design of the relief wells. The wells consist of a gravel-packed 8-inch diameter ID slotted wood stave well screen with 1/4 inch by 2-1/4 inch slots spaced 9 per every spiral. Plate 108 shows the relief well and the toe drain discharges, as related to reservoir and tail water elevations, for the period of 1954 to 1979. The relief wells were drilled by reverse circulation rotary methods using clear water. A few intervals in the highly pervious clean sands required the use of bentonite to stabilize the hole. After drilling and gravel packing, the holes were pumped clean, thoroughly surged and then test pumped for 4 hours. The yield from each well was variable, ranging from 30 gpm to 472 gpm, averaging about 292 gpm for the 54 wells. The average drawdown was about 10 feet. The contract for the wells was awarded to Layne Minnesota Company, 16 March 1953 and was completed by December 1953.

2.4 Seepage through the east abutment is controlled by the east abutment toe drains and lignite drains. These drains tap lignite beds which are the main paths of the seepage in the east abutment. See Plate 109 and 110 for location and profile of the drains. The drains consist of perforated clay pipe and corrugated metal pipe placed in interceptor trenches backfilled with bank-run gravel. The gravel was required to be well-graded, clean, durable and free of clay, but no processing was necessary. Although it was recognized that the lignite beds were probably more pervious than the bank-run gravel, the gravel was considered satisfactory as the lignites would be grouted or blanketed by impervious material upstream from the drains and, in addition, the primary function of the drains was to furnish relief against build-up of seepage pressure.

2.5 **SPILLWAY DRAINS**. Drainage for the spillway considered seepage through the lignite beds, leakage through joints and fractures in the floor slab and chute walls, and seepage beneath the crest structure in the contact between the structure and foundation materials.

2.5.1 **Crest Structure Drains**. The drainage for the crest structure is controlled by a subdrain beneath the structure, relief wells immediately downstream from the structure, and drainage behind the spillway abutments and chute walls.

2.5.2 The drainage beneath the crest structure is controlled by a 6-inch diameter, perforated, vitrified clay subdrain that is located immediately downstream from the crest structure cut-off key and runs the full length of the structure. A 6-inch diameter perforated header pipe extends out from the drain beneath each monolith to a 12-inch diameter, perforated,

vitrified clay collector pipe which also runs full length of the crest structure. All of the crest structure drains are founded in a clayey sand layer of the Fort Union and are all surrounded with screened gravel and sand filters. See Plate 111 for location of drains and design details. The screened gravel conformed to the following gradation:

Sieve Square Mesh	Total Percent Passing by Weight
3 inch	100
3/8 inch	15-55
No. 8	0-10
No. 200	0-2

Filter sand for the drains was obtained from Borrow Area B located on the downstream east bank and was required to be well-graded, clean, pervious sand conforming to the following gradation:

Sieve Square Mesh	Total Percent Passing by Weight
No. 10	50-90
No. 60	15-40
No. 200	0-10

2.5.3 Fourteen relief wells were installed, one each at a location 15 feet in front of every other spillway pier. All of the wells intersected the IU lignite and, in addition, six wells in the west one-half of the crest structure intersected the 1-1/2 U lignite. A discontinuous lignite stringer located at the east end of the crest structure was intersected by two relief wells. See Plate 112 for location and design details. Each relief well was cased with 6-inch diameter wood stave well casing having 1/4 inch by 3-inch slots spaced 7 per spiral. The slots were staggered in vertical rows. The wood staves were wound with No. 6 pipe winding wire. The casing couplings were fitted with a rubber gasket with a greased slip joint to allow for movement. Gravel pack for the relief wells was a mixture of washed sand and gravel graded to the following requirements:

Sieve Square Mesh	Total Percent Passing by Weight
5/8 inch	100
3/8 inch	75-90
No. 4	55-70
No. 16	30-45
No. 50	6-22

The Contractor was permitted to use any type of drilling equipment, which in the opinion of the Contracting Officer, was suitable to perform the work. If after drilling the hole, the lignite bed at the bottom of the hole was unstable and heaved into the hole, it was required that the hole be stabilized with sand and gravel before placing the well screen. On completion of drilling, the hole was required to be washed clean before placing the gravel pack, after which the well was lightly surged, bailed or pumped to clean the gravel pack. Following the surging, the annular spaced above the gravel pack was covered with a 12-inch layer of concrete sand and then backfilled with concrete up to the well pit. The main purpose of the relief wells is to reduce the pressure in the 1U lignite and preclude any excess uplift in the foundation of the crest structure and chute slab. This was necessary in the event the grout curtain became ineffective and the lateral drains for the 1U lignite in the chute area (to be discussed in a following paragraph) became clogged. The relief wells discharge into the crest collector drain which, in turn, discharges into the outfall drains running down the chute.

2.5.4 Drainage for the crest structure abutments is controlled by pervious backfill placed behind the abutment walls that intercepts seepage from the 2U lignite. A screened gravel blanket extends down to the base of the abutment slab and is connected to a rock drain. See Plates 113 and 114 for design of the abutment drains. Plate 115 shows a typical chute wall drain.

2.5.5 **Chute Drains.** The drains were designed to collect leakage through the concrete structures of the spillway, seepage through the lignite beds underlying the chute, seepage through cracks and joints in the slab, and relieve uplift pressures under the crest structure and chute paving. See Plate 116 for plan of the chute drains. Plate 117 shows the relationship of the lignite beds to the drain system. The system consists of a crest collector, outfalls and manholes, laterals, and lignite laterals. As many of the foundation materials beneath the chute slab are susceptible to frost heave, a 4-foot thick blanket of pervious fill was placed beneath the chute slab (previously discussed in paragraph 1.5.3, Chapter 8). The filter sand placed adjacent to the drainage pipes in the drain laterals was graded essentially as described in paragraph 2.5.2 of this chapter. The drain pipes were extra strength, perforated, vitrified clay pipe. All joints were made with a bituminous joint compound for flexibility. The pipes were bedded in 9 inches of filter sand, except the pipes for the lignite laterals and part of the west outfall which drain the 1-1/2U lignite. These intervals were bedded in screened gravel.

2.5.6 **Stilling Basin Drains and Relief Wells.** Lateral drains for the stilling basin were placed under the stilling basin chute slab. In addition, nine relief wells were drilled at the toe of the stilling basin chute slab and six relief wells were drilled beneath the stilling basin side walls. The side wall wells are referred to as shaft wells. See Plates 105 and 118 for location and design details. The design of the lateral drains is essentially as described for the spillway chute. The relief wells across the

stilling basin floor were gravel-packed 6-inch diameter slotted wood screens, also designed and drilled essentially as described for the crest structure relief wells. The shaft relief wells were somewhat different, having an unslotted wood pipe contained in grout above the screen and two 4-inch diameter refill pipes extending from the gravel pack to the floor of an access shaft to permit replacing any loss of gravel pack. The wells are joined to an access shaft by a 9-inch ID stainless steel sleeve in order to provide a durable joint capable of lateral motion. Above the sleeve, a 9-inch ID cast iron riser pipe extends to top of the access shaft to provide maintenance access. A cast iron tee was placed in the riser pipe line opposite a 24-inch wall outlet to the chute and stilling basin. In contrast, the slab relief wells discharge directly from concrete hoods into the stilling basin. See Plate 118 for relief well details. All the spillway drains are monitored with appropriate piezometers, which will be discussed in subsequent paragraphs.

2.6 **POWERHOUSE AREA DRAINS**. The major foundation drains for the powerhouse area consist of: weep holes through the surge tank and penstock portal walls that intercept a Fort Union sandy loam and the 1A and BB lignites (see Plate 119); weep holes in the powerhouse sump that intercept the 1B lignite (see Plate 102); and relief wells that intersect the 3B lignite, 4B lignite and lower sand horizon located at elevation 1400 (see Plate 121). In addition, the west abutment slope drains dewater the sand strata located in the upper portion of the west abutment as well as the 3A, 2A and X lignites (see Plate 122). Seepage from the west terrace gravel adjacent to the powerhouse area is dewatered by the switch yard subsurface drainage system (see Plate 123). Gradation of the filter sand, screened gravel and pervious fill for the above referenced drains was essentially as described for the spillway drains.

2.7 **POWERHOUSE RELIEF WELLS**. The initial powerhouse relief well installations consisted of five wells which were installed and capped before the embankment closure and river diversion. During diversion, the powerhouse area was flooded and the wells inundated. After the area was dewatered, the wells were checked and cleaned but no measurements of flow were made until 1957, at which time it was noted that most of the flow was coming from wells No. 1 and No. 5, which contacted the 1400 sand. As the relief wells had not relieved the pressures in the lignites and sand as was anticipated in the original design, four additional relief wells were installed in the powerhouse area during 1968 and eight additional relief wells were installed during the fall of 1970, after which a drop to acceptable levels was shown in the 3B and 4B lignites. See Plate 121 for location of the relief wells. Plates 156, 157 and 158 show location and design of relief wells completed during 1970.

3. DRAINAGE PROVISION SINCE CONSTRUCTION. In addition to the above referenced relief wells that were installed after completion of the project, a supplemental drainage system with relief wells was constructed in 1973 in the switch yard area. This additional drainage was required for the west terrace gravel after it was noted during 1965 that seepage had increased in

the west terrace gravel and the existing embankment toe drains and switch yard drains were not adequate to control it. The increase in seepage was apparently from the deeper gravels that were not intersected by the cut-off trench in the area west of embankment Station 30+00, or leakage through the sheet piling in this area. This assumption was made on the basis of pressure contours prepared for the west terrace gravel. Without remedial drainage it was feared that, if the cut-off trench failed to perform properly, piping could occur in certain silt horizons where the silts were excessive. See Plate 124 for the drainage plan, and Plate 125 for relief well design.

4. OTHER TREATMENT SINCE CONSTRUCTION.

4.1 **POWERHOUSE OUTLET CHANNEL SLIDE**. Minor small slides reoccurred in the unprotected west bank of the outlet works since an initial slide during construction in 1951 (refer to previous paragraph 2.18, Chapter 2). In addition to these minor slides, one notable slide has occurred in the riprap slope since construction. This slide occurred 26 May 1961 when 1 small portion of the riprap moved into the outlet channel along the right (west) bank of channel Station 166+00 to 167+00. For location of slope, see Plate 126. The slide was 100 feet long with about three feet of subsidence. Some minor subsidence had been noted in the slope for about a year prior to the slide. Inspection of the slide indicated that seepage in the area of the slide was coming from the 1A lignite. The observed seepage was about 30 to 40 gpm. A profile of the 1A lignite bed showed a sag in the plane of the bed that apparently ponded water. It was concluded that this seepage was the contributing factor to the slide of the riprap. The remedial treatment consisted of a drain at the top of the riprap slope in the road ditch abutting the 1V on 2H unprotected slope. The drain was installed 1 foot below the 1A lignite.

4.2 **SPILLWAY WEST SLOPE SLIDE**. The west cut slope of the spillway has experienced several shallow slope failures in the topsoil and Fort Union since completion of construction. The latest shallow slide occurred in 1979. The total length of this slide was about 350 feet (see Photo 55). These minor skin slides have been repaired by regrading.

4.3 **EMBANKMENT RIPRAP SLIDES**. Minor sloughing of riprap has occurred on the upstream slope of the embankment, generally 5 to 10 feet above the reservoir pool between elevations 1840 and 1845. The sloughs have usually been small varying from 20 to 35 feet in length and have occurred at embankment Stations 20+00, 21+25, 24+00, 24+50, 70+00, 123+00 and 125+00. All sloughs were repaired by 1977. Two types of riprap were used at the project. On the embankment face and a 300-foot reach of the left slope of the spillway approach channel, the rock pieces had a maximum size of 3/4 cubic yard with not over 10 percent less than 8-inch size. This rock gradation was referred to as Type A riprap; the rock was required to be well graded. For the remaining spillway riprap on the left slope, the largest allowable rock size was 8 cubic feet with not over 10 percent less than 6 inches. This rock was referred to as Type B riprap. Sources for the Garrison Dam riprap were from quartzite quarries near Elgin, North Dakota, fieldstone (glacial boulders)

from areas surrounding the damsite, and Sioux quartzite from the Sioux Falls, South Dakota area. The Type A protection consisted of one (1) foot of bank-run gravel filter, one (1) foot of spalls, and three (3) feet cf Type A riprap. The Type B protection consisted of one (1) foot of bank-run gravel filter, one (1) foot of spalls, and two (2) feet of Type B riprap.

4.4 **SLOUGHING POWERHOUSE WEST SLOPE**. Prior to May 1974, numerous small surface slumps occurred in the powerhouse west slope in the area immediately downstream from the west abutment slope drains (refer to paragraph 3.6 of this chapter for discussion of this drain system). Each sloughed area was generally associated with an exposed lignite bed that was emitting seepage. This situation was remedied by extending the west abutment slope drain system to drain the exposed lignite. Work on this extension was completed in 1978.

CHAPTER 10 - INSTRUMENTATION AND OBSERVATIONS

1. **GENERAL**. The instrumentation at the Garrison Dam consists of a complex system of piezometers, settlement gages, crest movement markers, consolidation gages, slope indicators and strong motion accelerographs. This chapter will briefly describe the instrumentation for the embankment and appurtenant structures and will identify any noteworthy results revealed by the respective instrumentations. Detailed information concerning the instrumentation at the Garrison Dam project is available from the Periodic Inspection Reports. See Plates 127, 128 and 136 for location of the piezometers, settlement gages, and slope indicators. The information that follows has been abstracted from the "Embankment Criteria and Performance Report," published in October 1981 and the Periodic Inspection Report No. 3, 1979. The instrumentation data are appropriate to these referenced report dates.

2. EMBANKMENT INSTRUMENTATION.

2.1 **PIEZOMETERS**. The piezometers for the embankment consist of wellpoint-type piezometers and U.S. Bureau of Reclamation double-tube piezometers. See Plate 129 for design of the wellpoint-type piezometers. The wellpoint piezometers were installed to measure the hydrostatic pressures in the valley alluvium, terrace gravels, abutment sands, and lignite beds. There are 136 wellpoint-type piezometers installed in the main embankment and foundation. A typical plot of the pressure observed for a gravel in the right valley foundation is shown on Plate 130.

2.2 The double-tube piezometers are installed in the embankment and foundation in four lines which include a total of 79 cells. The cells are connected by plastic tubes to a terminal well located near the downstream toe of the embankment. A typical plot of the readings for the double-tube piezometers is shown on Plate 131. The purpose of the double-tube piezometers was to monitor pore pressure buildup during construction and during consolidation.

2.3 **SETTLEMENT GAGES**. The settlement gages have two basic designs: a plate-type gage that was installed directly on the foundation surface with the riser pipe extended as the embankment was placed; and a cross arm gage that has cross arms at 5-foot vertical intervals on a vertical pipe. The plate type has its lower end perforated and surrounded with gravel pack so that it can also function as a piezometer. The cross arm type were designed to reflect both magnitude and rate of foundation settlement as well as consolidation in the embankment. Originally, there were 27 plate-type gages and 12 cross arm gages, but nine of the plate type and three of the cross arm type were inundated by the lake and are no longer operative. Plots of settlement versus fill height are shown on Plate 132 and consolidation versus fill height are shown on Plate 133. The largest estimated settlement along the centerline of the embankment is 9.4 feet at Station 49+00.

2.4 __CREST MOVEMENT MARKERS__. The crest movement markers consist of monuments installed on a line across the crest of the embankment. The monuments are surveyed periodically and referenced to fixed monuments in the abutments to determine horizontal and vertical movement in the embankment. See Plates 134 and 135 for location of markers and plots of the vertical and transverse embankment movements.

2.5 __SURFACE CONSOLIDATION GAGES__. There are eight surface consolidation gages installed adjacent to the plate-type settlement gages. These gages are similar to the crest structure monuments. The consolidation of the embankment is determined by the difference in settlement of the surface gage with that of the plate on the settlement gage. The consolidation since the end of construction in 1954 is about 1.0 feet.

2.6 __SLOPE INDICATORS__. The slope indicators consist of a quad-gooved 3.19 inch OD extruded epoxy resin aluminum casing placed in a 6-inch diameter or larger hole. The space around the outside of the casing is backfilled with sand. Four self-aligning grooves equally spaced around the inside circumference of the casing orient the reading instrument as it is guided down the hole by the grooves. The reading instrument is a Digitilt bore-hole probe Model 025 manufactured by the Slope Indicator Company, Seattle, Washington.

2.7 There are two slope indicators located on the embankment at Station 20+00. These slope indicators were installed in 1971 and 1972 to monitor movements in the embankment that were noted in a settlement gage during 1968, 1969 and 1970. Since installation of the slope indicators there has been very little movement at the referenced station. In addition to the slope indicators in the embankment, there are two slope indicators located in the right cut slope of the powerhouse excavations, one slope indicator at the intake structure abutment, and one slope indicator along the left (east) slope of the spillway. These slope indicators have not shown any significant movements. For location of slope indicators, see Plate 136.

2.8 __STRONG MOTION INSTRUMENTATION__. Instrumentation to record seismic events at the Garrison Dam consist of three Kinemetric SMA-Strong Motion Accelerographs having the capability of recording seismic activity in the range of 0 to 0.1g, with starting acceleration of 0.01g. Data are recorded on 70mm photographic film. Each instrument is powered by solar panels. The instrumentation is located at Station 85+00 near the embankment crest, in the penstock area between the downstream surge tanks and powerhouse, and 2200 feet downstream of the embankment in the fish hatchery building. All data are checked every four months by USGS personnel under an agreement administered by the Waterways Experiment Station in Vicksburg, Mississippi. In accordance with the Standard USGS Seismic Risk Map of the United States, the Garrison Dam is located in Seismic Risk Zone 1. In this zone, it is considered that minor damage may be caused to structures by distant earthquakes. The intensities for the area correspond to V and VI on the Modified Mercalli-Intensity Scale.

2.9 **SPILLWAY INSTRUMENTATION.** Instrumentation for the spillway is comprised primarily of piezometers to monitor the seepage pressures in the numerous lignite beds beneath the crest structure and spillway sidewalls, as well as the concrete-bedrock contacts beneath the weir. See Plate 127 for location of piezometers. Other instrumentation includes the numerous structural movement inserts embedded in the concrete to monitor movements of the crest structure, spillway slab, and walls.

2.10 The piezometers installed in the lignites show that the grout curtain immediately upstream from the crest structure is effective in reducing seepage pressure in the lignite beds and is becoming more effective with time. The piezometers also show the effectiveness of the spillway relief wells in relieving seepage in the 1L lignite. Piezometers at the weir-bedrock contact have not shown any specific pressure pattern probably because of the general impervious nature of the Fort Union. It was stated during early observation of these piezometers that it might take years to establish a final pressure pattern in this area. Recent changes in some of the crest piezometer readings suggest that the hydrostatic effects of the lake water may have finally reached the crest area. See Plates 137 through 140 for spillway piezometer curves.

2.11 Measurements of the movement inserts in the spillway show that the spillway features are still in a state of rebound, but the rebound is decreasing. Rebounding of the crest structure has been more pronounced in that portion of the structure located north and west of the SF-1 fault. The maximum rebound of the crest structure is 0.587 feet at monolith 12. The largest rebound for the chute slab is about 300 feet downstream from the crest structure in the area north of the SF-1 fault. This area has a rebound of 1.223 feet. Maximum rebound for the chute walls is 0.6 feet. See Plates 141 and 142 for rebound curves and contours.

3. **POWERHOUSE AREA INSTRUMENTATION.** Piezometers in the powerhouse area are designed to monitor pressures in the lignite beds and sand layers. See Plates 119 through 122 for location, sections through lignite and sand beds, and piezometer curves. In addition to these piezometers, four piezometers have been installed in powerhouse Unit 2 to monitor uplift pressures. These piezometers have shown little change in hydrostatic pressures and show pressures well below the tailwater elevation.

3.1 Movement measurements from the movement inserts embedded in the structures indicate that the powerhouse and stilling basin are still rebounding but the surge tank bays have settled. Rebound for the powerhouse and stilling basin is between 0.3 feet and 0.7 feet and is proportional to the amount of overburden removed. Maximum settlement in the surge tank area is 0.1 feet. See Plate 143 for curves showing typical vertical movement. The intake structure settlement is about 0.55 feet. The tunnel reflects both uplift and settlement. The largest settlement occurs at the intake structure and decreases uniformly downstream until it rebounds. See Plate 144 for vertical movement profile of Tunnel No. 1. The maximum settlement for the tunnels occurs in Tunnel No. 1 which has a settlement of 0.28 feet. The maximum rebound is 0.1 feet.

CHAPTER 11 - ADDITIONAL FEATURES OF THE PROJECT

1. SNAKE CREEK EMBANKMENT.

1.1 **GENERAL**. The Snake Creek embankment is located about 8 miles northeast of Garrison Dam on an arm of the Garrison reservoir that is included in the valley of Snake Creek. See Plate 145 for location. See Photo 56 for aerial view of embankment. The reservoir formed by the embankment is a sub-impoundment for wildlife and recreation and is a diversion for water from the Garrison reservoir into The Bureau of Reclamation's Missouri River Diversion project, which will eventually divert water to the James and Sheyenne Rivers for irrigation purposes. The embankment also serves as a relocation for U.S. Highway 83 as well as for a railroad and utility facilities. The project was constructed during 1951 and 1952 under one contract. Included in the embankment is a low-level regulating conduit which permits flows between the Garrison reservoir and the Snake Creek pool, and a pumping plant to pump water from the Garrison reservoir into the Snake Creek pool. See Plates 146 through 149 for plan, profile and embankment section.

1.2 **EXPLORATIONS**. Foundation investigations were made with churn drills that recovered continuous 6-inch diameter drive samples of the overburden, and rotary drills that recovered 5-3/8 inch diameter cores of the bedrock. A test pit and several power auger borings were made for borrow investigations and resistivity surveys were made to locate pervious materials in the areas adjacent to the embankment. See Plates 146 though 148 for the boring plan. A tabulation of the subsurface investigations is shown below:

TABLE 8

TABULATION OF SUBSURFACE INVESTIGATIONS
SNAKE CREEK EMBANKMENT

Exploration Method	No. of Explorations	Purpose of Exploration
Boring	74	Embankment foundation, pumping station and channel, relief well contract.
Boring	11	Impervious Borrow
Boring	26	Pervious Borrow
Boring	24	Alternate Sites
Total Borings	135	
Power Auger	17	Pervious Borrow
Test Pit	1	Pervious Borrow
Restivity	192 stations	Pervious Borrow

1.3 **GENERAL GEOLOGY**. The major topographic feature at the site is the broad glacial valley which extends 2-1/2 miles between the north and south abutments. The ground surface is hummocky and rises and falls in a series of knob and kettle forms that are typical of glacial moraine topography. Major drainage within the valley is through Snake Creek, a post-glacial stream which has incised its valley approximately 100 feet below the north abutment and aggraded a flood plain which varies in width from 150 to 1500 feet. A deep glacial valley has been eroded at least 350 feet into the Fort Union bedrock. The Fort Union bedrock supports the glacial deposits in the area.

1.4 **FOUNDATION OF THE EMBANKMENT**. The foundation strata for the embankment are comprised of a sequence of glacial deposits ranging from sand and gravel to glacial till. See Plate 150 for the geologic profile. The deposits in ascending order are: 1) lower sand and gravel, 2) lower glacial till, 3) upper sand and gravel, 4) north abutment sand and gravel, 5) remnant outwash, and 6) upper glacial till. A brief description of each follows:

1.4.1 **Lower Sand and Gravel**. The lower sand and gravel fills the lower valley and rests upon Fort Union clay-shale. In the area immediately below the lower glacial till the lower sand and gravel includes outwash materials which are stratified with lenses of glacial till. Sections of sandy silt and clay are found interbedded throughout the lower portion.

1.4.2 **Lower Glacial Till**. The lower till consists of stiff, sandy clay with about 10 percent pebbles and cobbles. It varies from 0 to 70 feet thick.

1.4.3 **Upper Sand and Gravel**. The upper sand and gravel extends completely across the valley except for a small interval of glacial till in the central part of the valley. The horizon consists predominately of fine to coarse sand and silty sand in its upper reaches and gravelly sand below.

1.4.4 **North Abutment Sand and Gravel**. The sand and gravel blankets the north abutment and is covered with 20 to 50 feet of glacial till. It consists primarily of coarse sand and gravel with silty zones ranging from 10 to 30 feet thick.

1.4.5 **Remnant Outwash**. These deposits appear to be fillings in temporary channels for glacial melt water. The deposits range from 1 to 5 feet in thickness and consist predominately of fine sand and silt with some gravelly sand and clay.

1.5 **BEDROCK.** The Fort Union is the bedrock throughout the area of the embankment. The estimated total thickness of the formation at the site is about 1,000 feet. The regional dip of the Fort Union is westward but local irregularities and small structures can be noted in the gentle undulations of the lignite beds. No system of jointing was observed at the site although joints were apparent in the lignites which were recovered in rotary core samples. Some slickensides were noted in the fatter clays but

they did not appear to indicate any marked degree of movement. The Fort Union at the Snake Creek embankment has essentially the same lithologic characteristics as at the Garrison Dam.

1.6 **RELIEF WELLS**. The purpose of the Snake Creek embankment relief wells is to reduce the excessive underseepage pressure on the Garrison reservoir side of the embankment when water is held in the Snake Creek pool for sub-impoundment. The relief wells at Snake Creek are normally submerged by the Garrison reservoir and are not accessible for observation.

When the Snake Creek pool is higher than the Garrison reservoir, the relief wells will be in operation. However, neither the relief wells nor the areas of questionable overburden cover are exposed until the Garrison reservoir drops below elevation 1783. Because of this condition, the design of the relief wells was much more conservative than for wells that are not submerged and can be closely monitored. It was originally thought that the surface sand beneath the embankment would be completely intersected by the cut-off trench but when the cut-off trench was excavated it could not be determined if this had been accomplished. To alleviate this doubt, the impervious waste blanket over the swamp areas was extended to cover the exposed surface sands and ditch drains were added. See Plate 150 for location of relief wells, cut-off trenches and blanket. The relief wells were drilled and designed essentially as described in preceding paragraphs concerned with the Garrison Dam embankment relief wells. See Plate 151 for design of relief wells.

1.7 **INSTRUMENTATION**. Instrumentation for the Snake Creek embankment consists of 13 wellpoint-type piezometers of the same design as previously described for the Garrison Dam embankment. In addition to the piezometers, there are 32 crest movement markers. All readings from these installations indicate that the embankment is performing adequately.

2. PROTECTIVE WORKS CITY OF WILLISTON.

2.1 **PURPOSE OF PROJECT**. The purpose of the Williston project is to protect the City of Williston and the facilities of the Great Northern Railway from floods in the backwater reaches of the Garrison reservoir. The protection consists of a levee about 9 miles long, two pumping stations to handle interior drainage, a water intake and sewage lift station, and a strengthened railway embankment to resist flood waters on both sides. See Plate 152 for the general plan. Williston is located in Williams County, northwestern North Dakota, about 150 miles upstream from Garrison Dam. In addition to the work at Williston, minor bank stabilization and some drainage treatment was made at the Buford Trenton and Lewis and Clark Irrigation Districts located a few miles upstream from the Williston area, and also at curve No. 9 of the Great Northern Railway that adjoins the Garrison reservoir a short distance upstream from Williston.

2.2 **GENERAL GEOLOGY**. The surficial geology at the Williston area reflects the characteristics of continental glaciation. However, in the Missouri River and Little Muddy Creek valleys, it is difficult to differentiate the glacial materials from the post-glacial deposits because of the reworking effects of post-glacial water action. For this reason, all foundation materials for the Williston levee project have been classified as alluvial deposits only. See Plates 153 and 154 for a physiographic map and generalized geologic profile along the centerline of the levees.

2.3 **INVESTIGATIONS**. Subsurface investigations for the Williston project were conducted during 1952 and 1953 at which time 6-inch diameter churn drill borings and auger borings were made to determine foundation conditions. Continuous drive samples were taken in the glacial overburden and where Fort Union bedrock was encountered, the sampling was usually limited to a 5- or 10-foot drive sample into the bedrock. The number of strokes per 1-1/2 foot drive were recorded during the churn drilling. The data was used in conjunction with the test data obtained on undisturbed samples to determine physical properties of soft foundation materials. Where accessibility limited the use of the churn drill, a 6-inch diameter hand auger was used for sampling purposes. Soft materials were recovered with a 3-inch diameter Osterberg sampler operated from the churn drill or portable tripod.

2.4 **FOUNDATION MATERIALS**. The foundation material for the levee and appurtenant structures is comprised of alluvial sand and gravel which is generally mantled with a silt and clay blanket. The thickness of the blanket varies along the levee alignment but is usually the thickest in backfilled channels and in the Muddy Creek terrace. The sands and gravels beneath the natural blanket are comprised of predominantly silty fine sand and fine sand with discontinuous gravel horizons. See Plate 54 for the levee foundation materials. The Fort Union bedrock underlies the alluvial materials and is very similar to the Fort Union at the Garrison Dam, consisting of flat-lying, partially indurated deposits of gray clay (clay-shale), brown to grayish fine sands, and lignite beds. The clays and sands are discontinuous, often cross-bedded and vary from thin partings to beds over 15 feet thick.

2.5 **ENGINEERING PROPERTIES OF FOUNDATION MATERIALS**. Testing for the engineering properties of the foundation materials was limited to samples from the marsh and slough areas as the routine boring information indicated the other foundation materials were adequately strong for the levee construction. Information concerning the test data is shown on Plate 155.

2.6 **RELIEF WELLS**. The need for relief wells along the levee alignment was based on a minimum gradient factor of safety of 1.25 against uplift at the toe of the land side of the levee berm. Seventy-three wells were installed with each well designed for 50 percent penetration. The wells were constructed during 1959 and 1960 by the Layne-Minnesota Company of Minneapolis, Minnesota. The contractor used a Caldwell Auger rig with a 36-inch bucket to drill the upper 6 to 20 feet of the natural blanket materials, and the underlying sand and gravel was drilled with a Williams

Rotary rig using straight rotary methods. The wells were constructed of 8-inch inside diameter, slotted wood stave screen surrounded with 6 inches of gravel pack. The slots were 1/4-inch in width and were spaced to result in 30 square inches of clear slot openings per lineal foot of screen. The riser pipe consisted of 8-inch diameter asbestos-bonded corrugated metal pipe attached to the upper part of the screen and contained in grout above the water table. Drilling practices and development were routine, essentially as described for the Garrison Dam relief wells. Piezometer tubes were installed along the line of relief wells to monitor piezometric uplift pressures during flood periods. No unusual foundation conditions were noted during construction of the levees or relief wells.

CHAPTER 12 - POSSIBLE FUTURE PROBLEMS

1. **SUMMARY**. There have been no major changes or contemplated changes to the Garrison Dam since construction. The discussions herein will be concerned primarily with the minor continuing problems that have existed at the project since it became operational, such as slope erosion, drainage problems, and minor structural movements. Based on the latest observations as presented in the Garrison Dam Periodic Inspection Report No. 3, 1979, there are no serious deficiencies of the embankment, powerhouse, tunnels, intake structure, and spillway that present a serious problem to the integrity of the project. No unusual movements or instabilities are occurring and all facilities appear to be functioning in accordance with their designed purpose.

2. **SEEPAGE AT THE POWERHOUSE WEST SLOPE**. Ground water seepage from the 3A, 2A and X lignite beds in the powerhouse west slope is controlled by an appropriate drainage system. These drains have been successful in controlling the drainage, but the area is a continuous problem in respect to surface sloughing. This occurs where the lignite seepage softens the surface material and causes surficial sloughing and slumps, especially below the slope berms. Close observation of these drains is necessary to assure satisfactory performance. Future modifications may be necessary to eliminate the sloughing problems.

3. **SWITCHYARD DRAINAGE SYSTEM**. The switchyard drainage system intercepts and drains the west terrace gravel. Increases in drainage from this area prompted the installation of additional drains and relief wells in 1972 and 1973. It was believed the increase in drainage was due to underseepage in gravel deposits below the sheet pile cut-off or through portions of the sheet piling that may have been displaced during construction. This interval is in the general area of Station 30+00. Close monitoring of the area will be necessary to assure that no pressure buildups take place that could result in foundation piping.

4. **RELIEF WELLS**. The relief wells will require continuous remedial treatment including cleaning, surging, and periodic pumping to assure peak performance. Recent inspections show water in many places has backed into the outfall pipes and into the relief wells during high water in the discharge channel. The high water appears to be caused by beaver dams and siltation. Also, many flap valves for the outfalls are missing and discharge pipes are often filled with debris and vegetation. These are minor deficiencies, but require continuous maintenance to permit the wells to function effectively.

5. **SPILLWAY SURFACE SLIDES**. The surface slides on the right cut slope of the spillway as discussed in previous paragraph 4.2, Chapter 9, will undoubtedly continue to occur. The slope was last repaired in 1978 and additional sliding occurred during the spring of 1979. Lateral drains may be required throughout the unstable areas to alleviate this condition.

6. MOVEMENT OF EMBANKMENT RIPRAP. Minor sloughing of the upstream riprap has occurred on the embankment slope at Stations 20+00, 21+25, 24+00, 24+50, 70+00, 123+00 and 125+00, the downward slumping of the riprap has usually occurred 5 to 10 feet above the pool elevation. However, the slumps have not exposed the embankment materials. The affected areas are 20 to 35 feet wide and are generally between elevations 1840 and 1845. The areas were all repaired by 1977. It is expected that additional slumping will occur and will require rearrangement of the rock materials. A large supply of riprap material is available from stockpiles on the west abutment.

7. UNUSUAL VERTICAL SETTLEMENT OF EMBANKMENT. Vertical settlement of the embankment departed from the normal in the area of Station 20+00, right (west) abutment. The departure in the settlement gage readings occurred during the period from 1964 to 1970. The readings showed that this area settled more than normal and the foundation settlement exceeded the average crest settlement. Studies of this condition indicated that the unusual movement was in the foundation rather than in the embankment. Some of the vertical movement appears to have been absorbed by the bridging action of the embankment. It was thought that the movement could have been caused by consolidation of the colluvial clay and silt slopewash which comprise the embankment foundation materials in this area. These deposits are shown in section along the centerline of the dam on Plate 40. Since 1970, the rate of movement appears to be about normal. Additional instrumentation has been installed in this area and continues to be closely monitored. Detailed information concerning the settlement is available from the Garrison Dam settlement and consolidation studies report, Omaha District, September 1975.

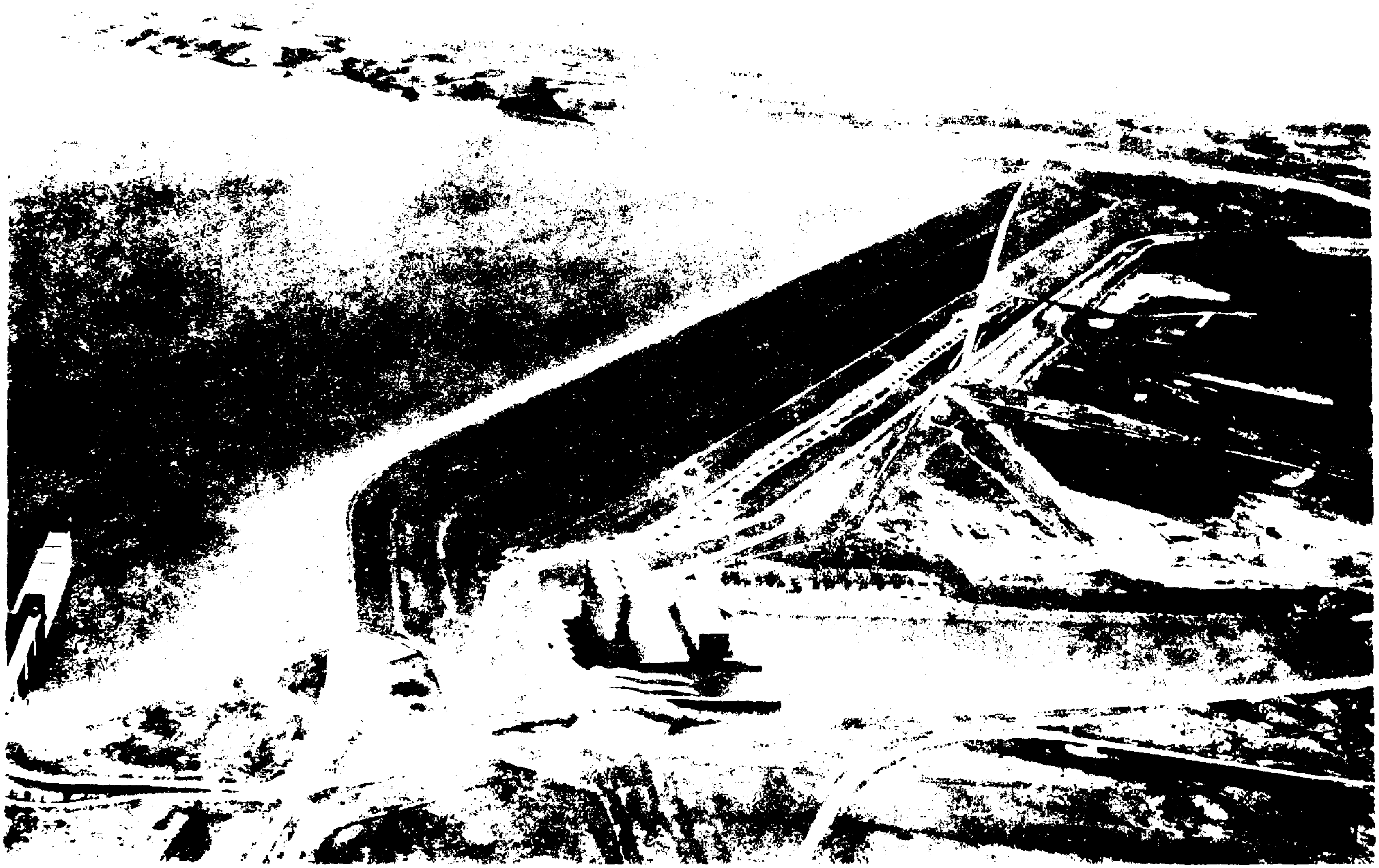

PHOTO NO. 1 Aerial view of Garrison embankment and appurtenant structures

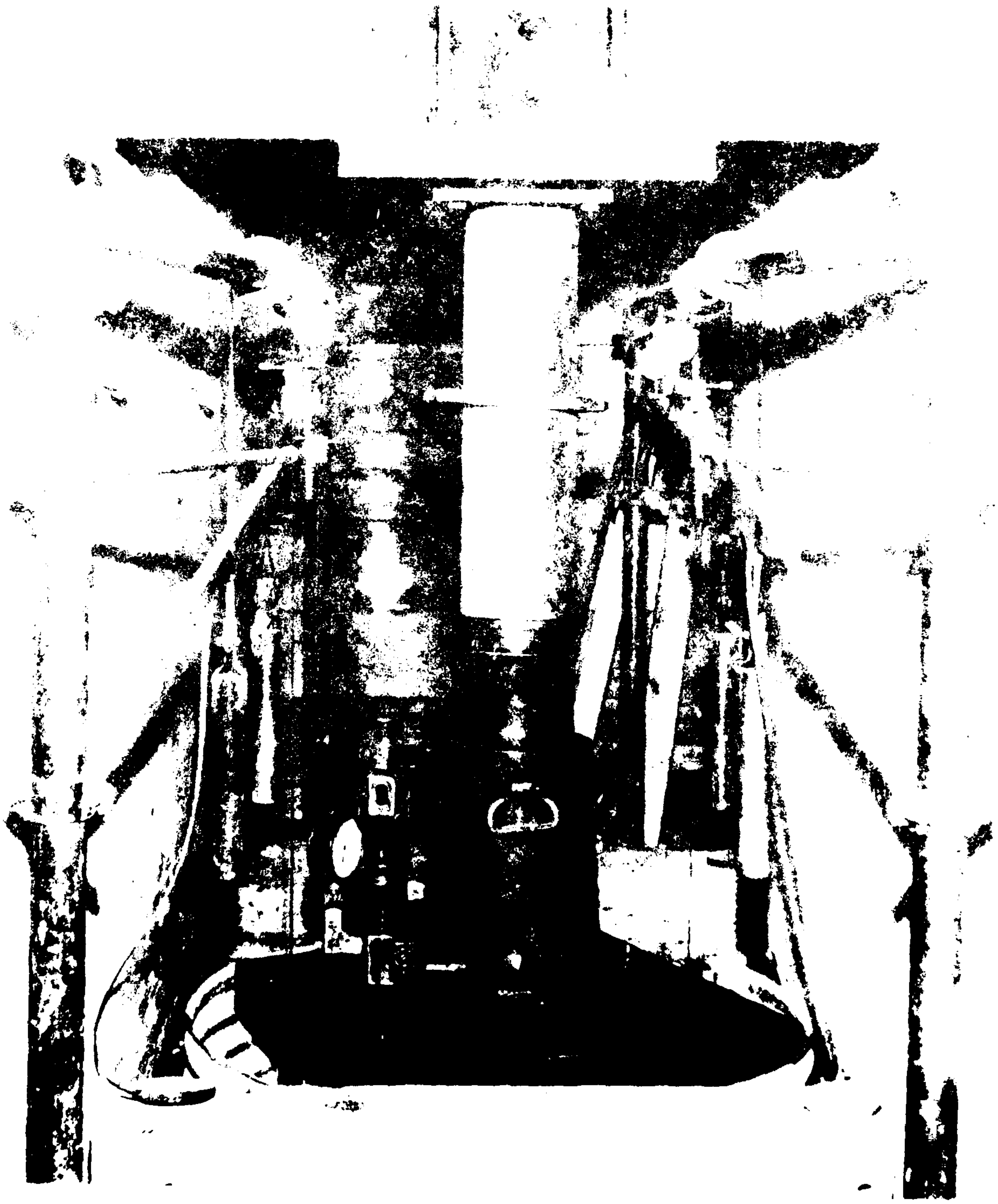

PHOTO NO. 2 View of plate bearing test equipment

PHOTO NO. 3 View of cooling tank and circulating pump.

PHOTO NO. 4 View of freezing chamber being lowered into hole

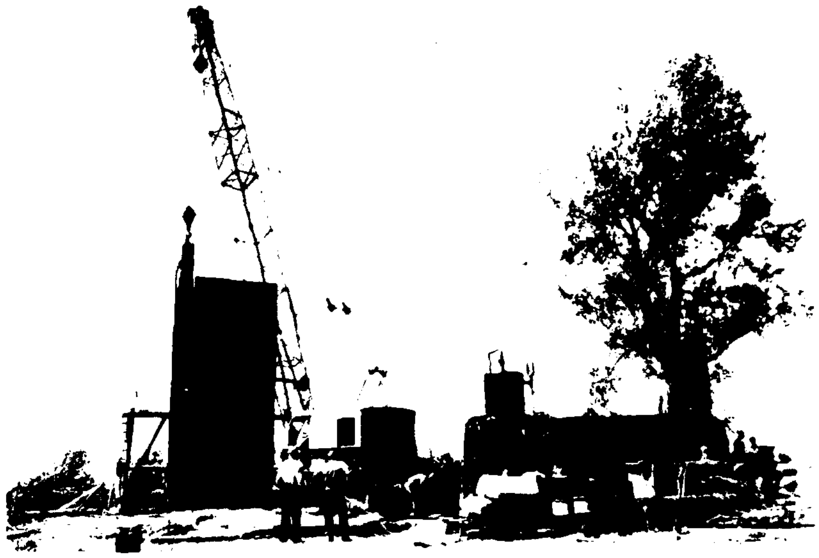

PHOTO NO. 5 View of test pile driving operations

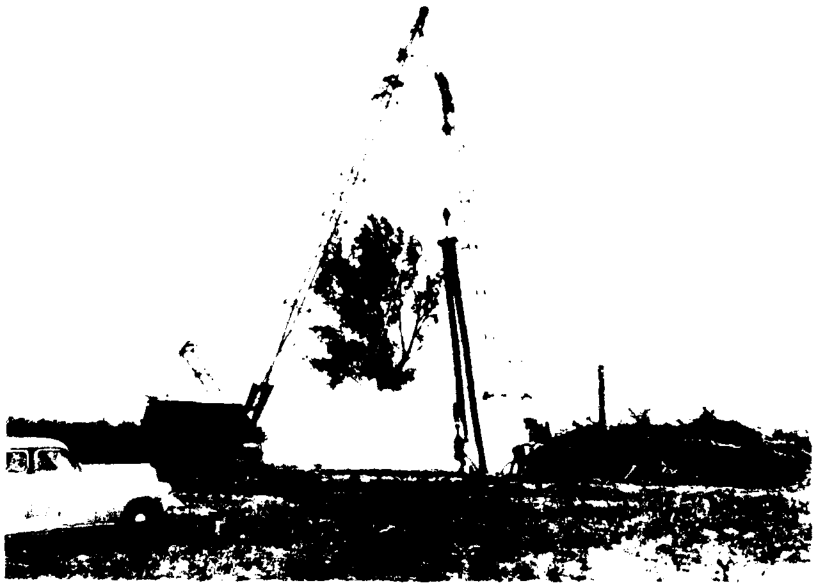

PHOTO NO. 6 View of test pile extraction operation

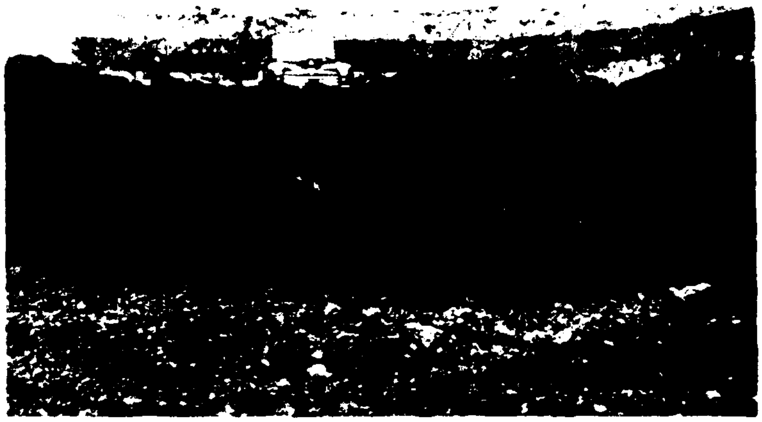

PHOTO NO. 7 View of grout in face of saw cut BB Lignite

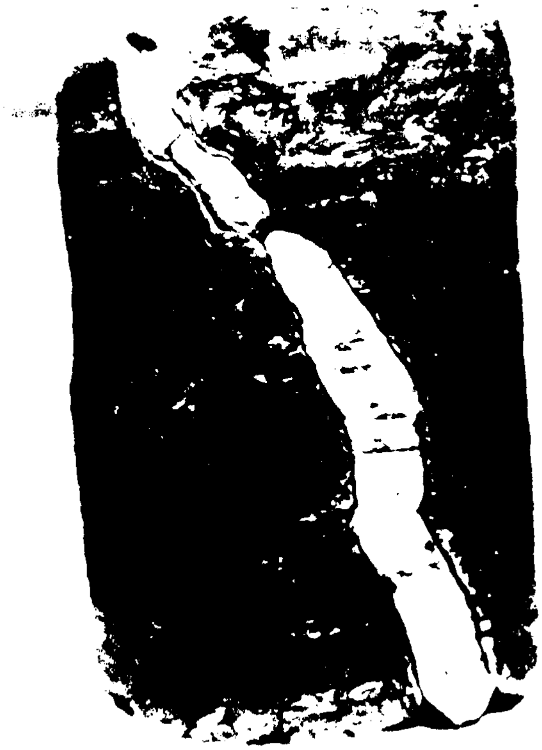

PHOTO NO. 8 View of grout in core of Lignite

PHOTO NO. 9 View of test grouting equipment

PHOTO NO. 10 View of rock salvage pile

PHOTO NO. 11 View of rock samples for weathering test

PHOTO NO. 12 View of movement in powerhouse foundation showing crack and exploration trench

PHOTO NO. 13 View of upstream face of upstream key showing small folds

PHOTO NO. 14 View of saw cut downstream
face of powerhouse excavation

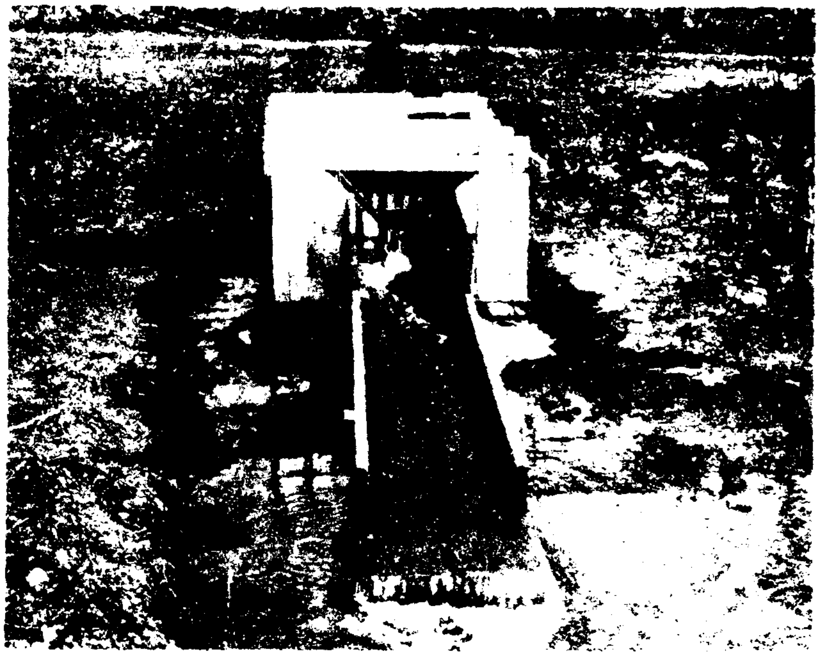

PHOTO NO. 15 View of weir box

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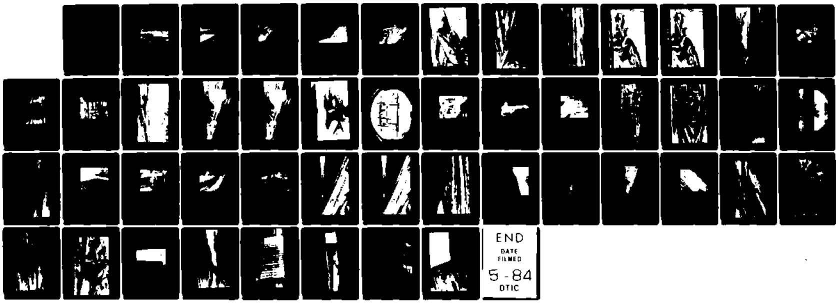

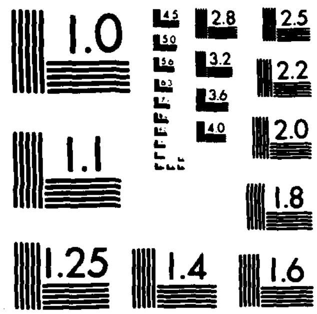

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NATIONAL BUREAU OF STANDARDS-1963-A

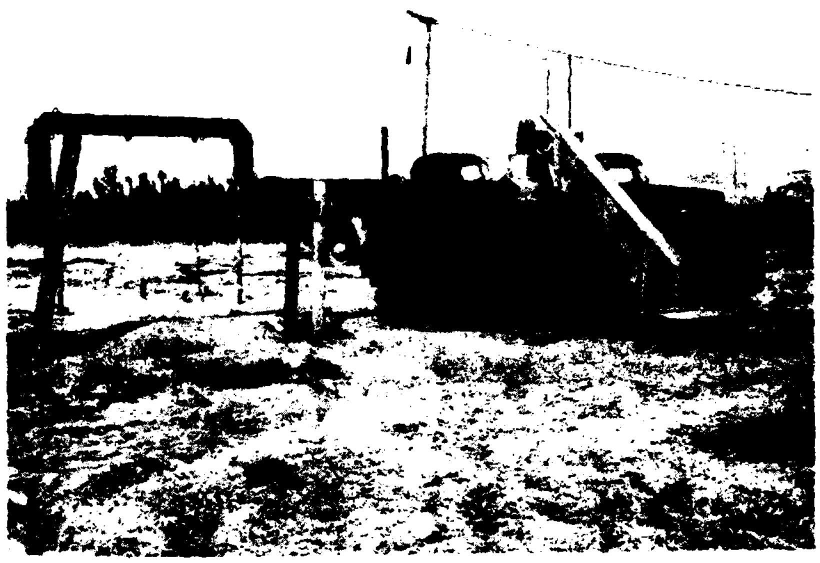

PHOTO NO. 16 View of pumping equipment

PHOTO NO. 17 Front view of intake channel slide

PHOTO NO. 18 Side view of intake channel slide

PHOTO NO. 19 View of outlet channel slide

PHOTO NO. 20 Close-up view of outlet channel slide

PHOTO NO. 21 View of excavations for spillway

PHOTO NO. 22 View of excavations for spillway approach channel

PHOTO NO. 23 View of spillway stilling basin excavations

PHOTO NO. 24 View of intake and powerhouse excavations

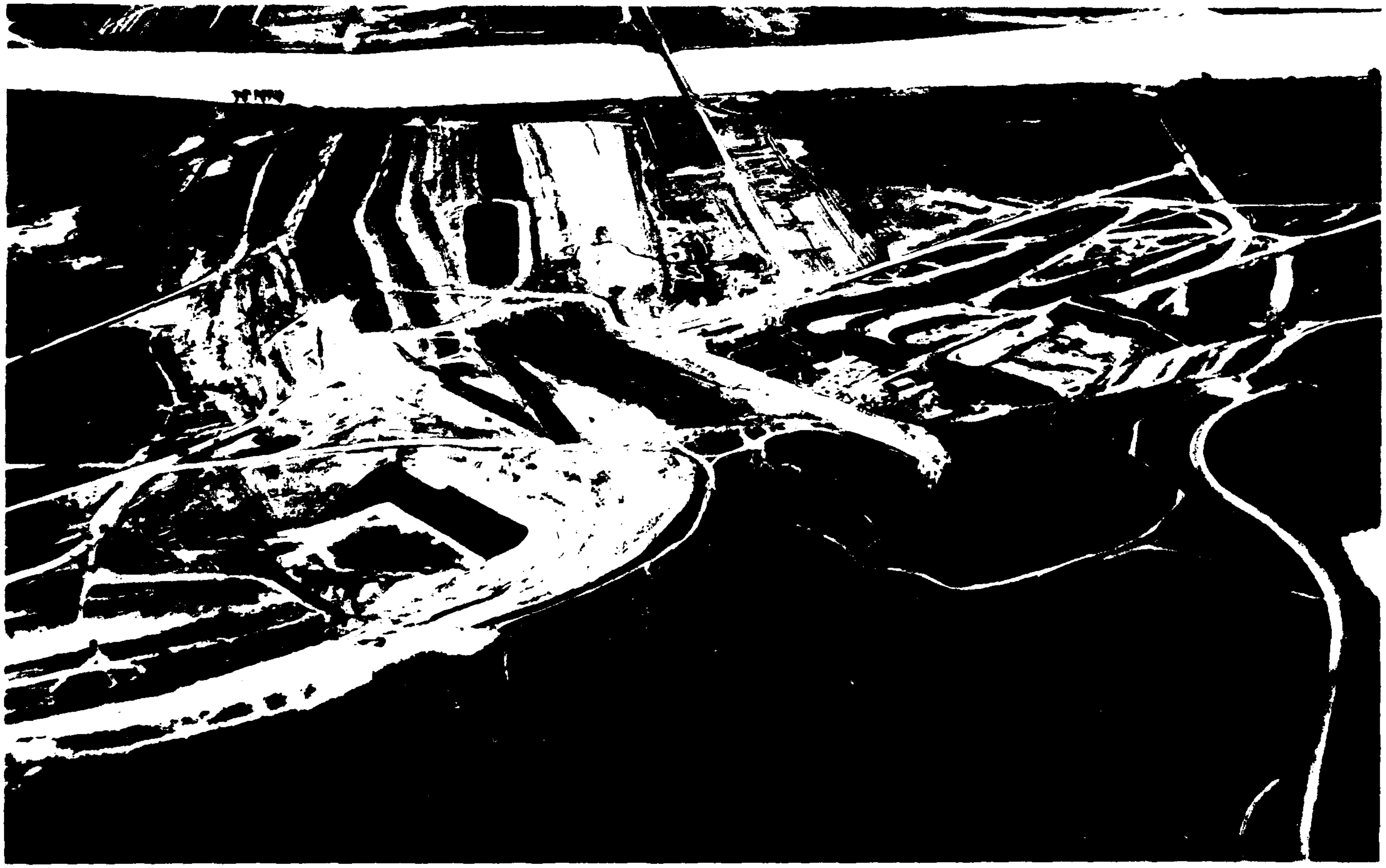

PHOTO NO. 24 View of intake and powerhouse excavations

PHOTO NO. 25 View of Lignite salvage operations in outlet channel.

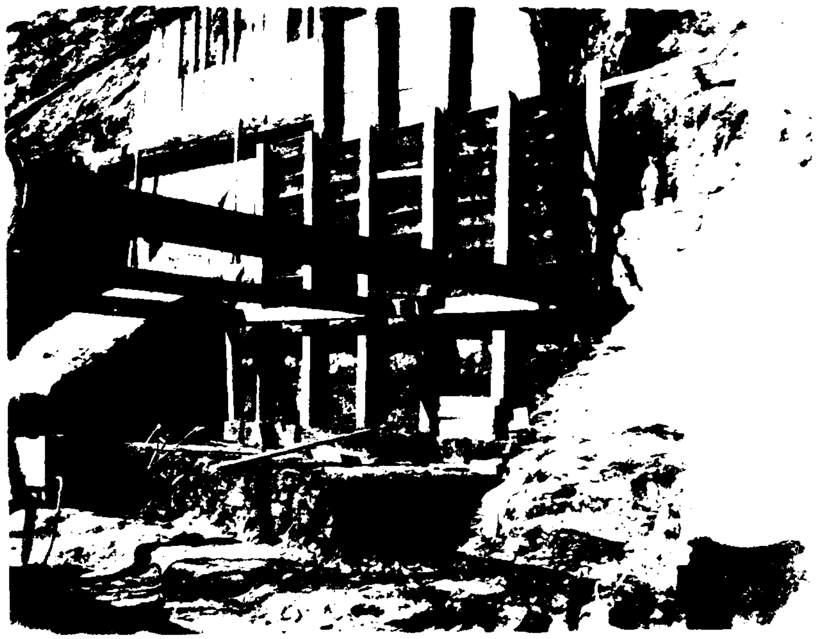

PHOTO NO. 26 View of temporary shoring downstream portals

PHOTO NO. 27 View of wood bracing upstream portals

PHOTO NO. 28 View of corrugated metal sheets to control sloughing from saturated vertical excavation upstream portal

PHOTO NO. 29 View of cut-off trench and sheet piling construction left center

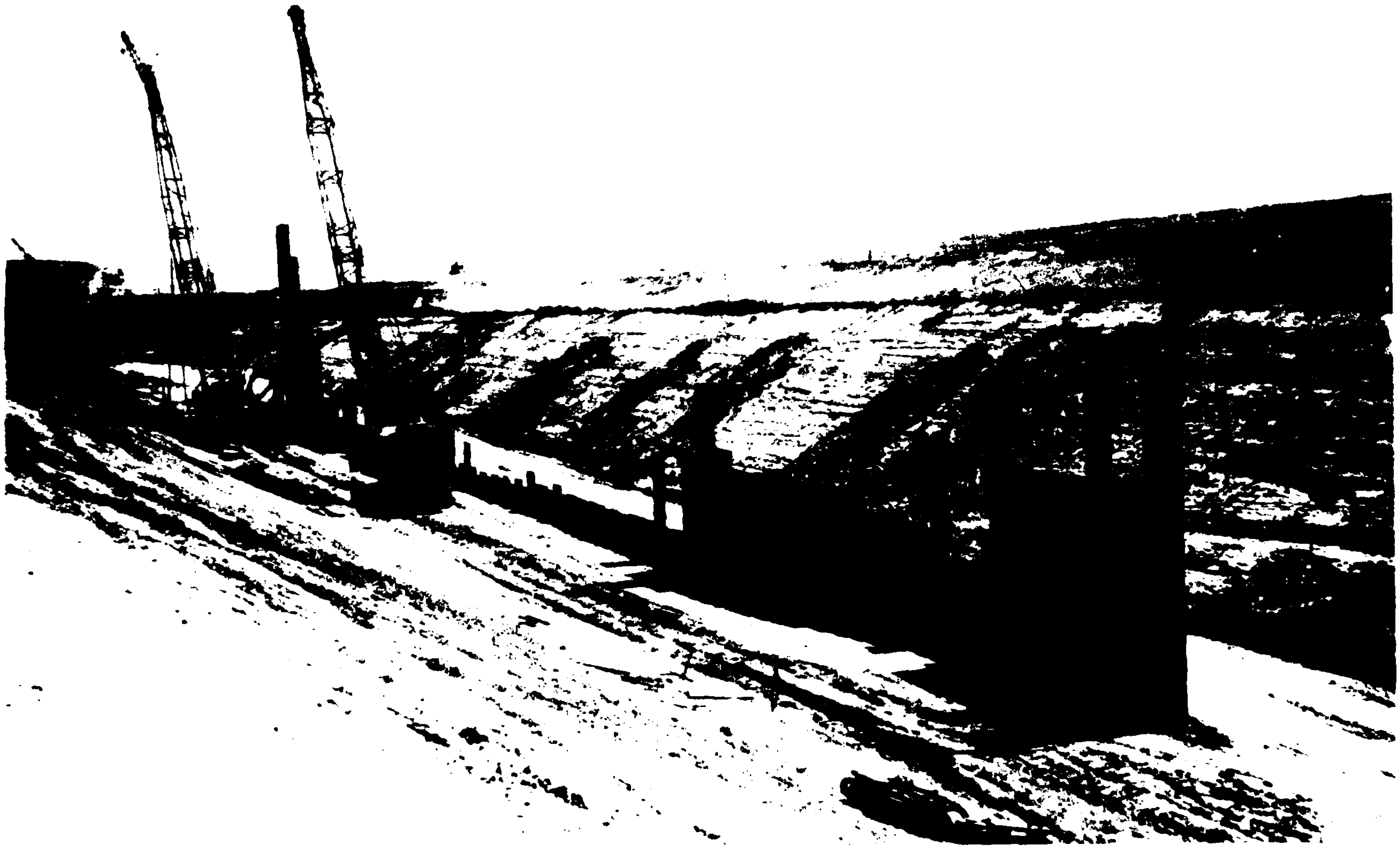

PHOTO NO. 30. View of sheet pile driving in cut-off trench.

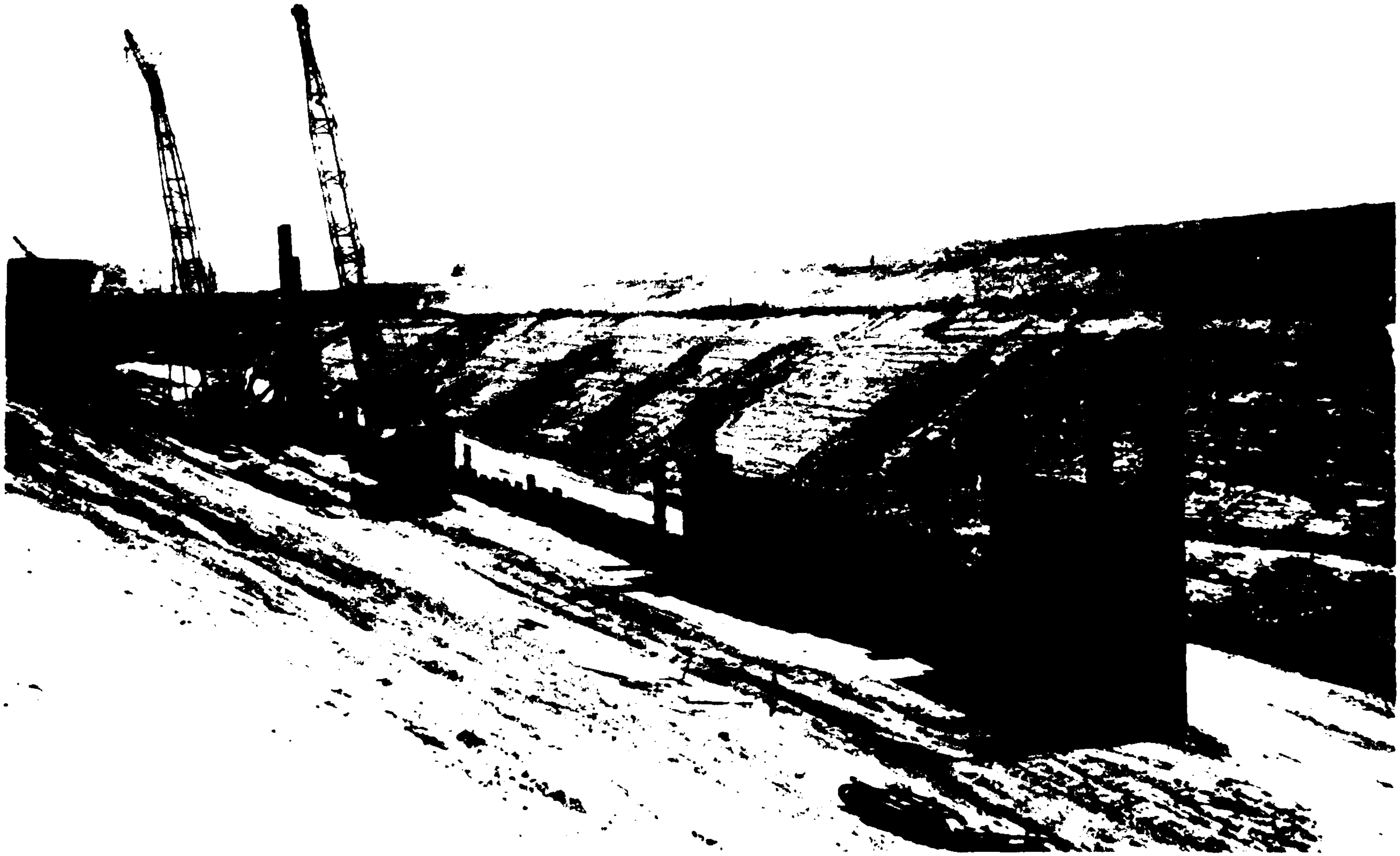

PHOTO NO. 30. View of sheet pile driving in cut-off trench.

PHOTO NO. 31. View of Stage I excavations at start of test tunnel construction

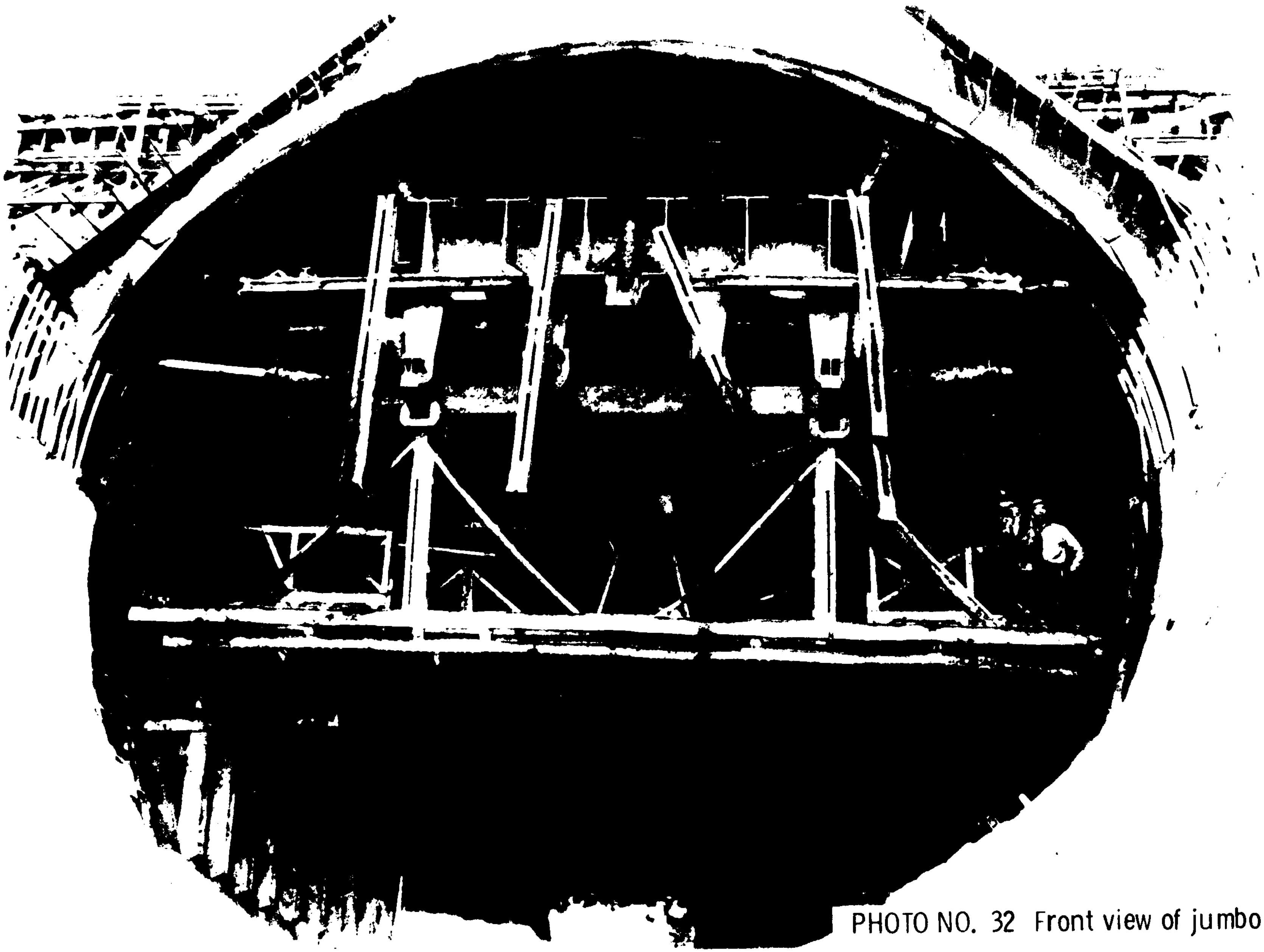

PHOTO NO. 32 Front view of jumbo

PHOTO NO. 33 Rear view of jumbo

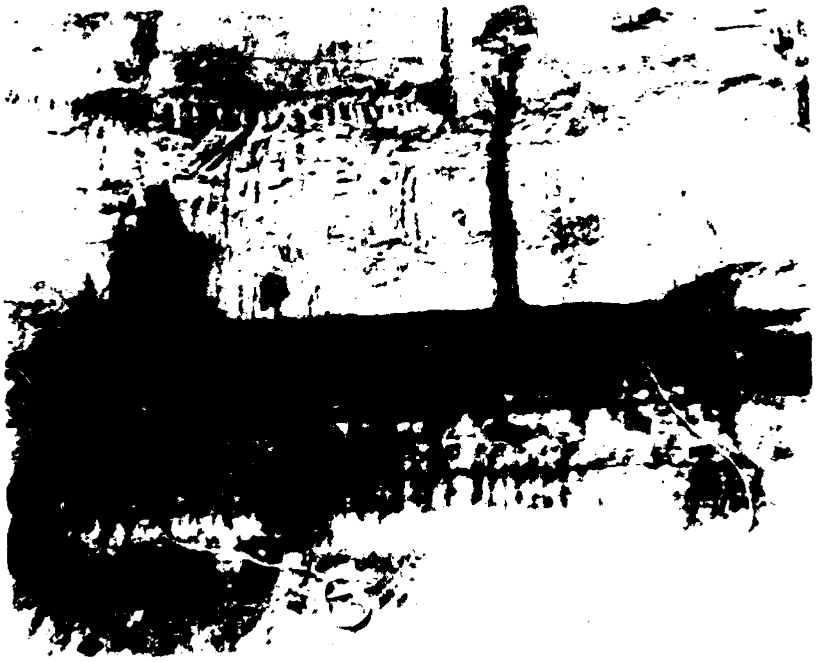

PHOTO NO. 34 View of downstream portals showing steep slope 1A Lignite

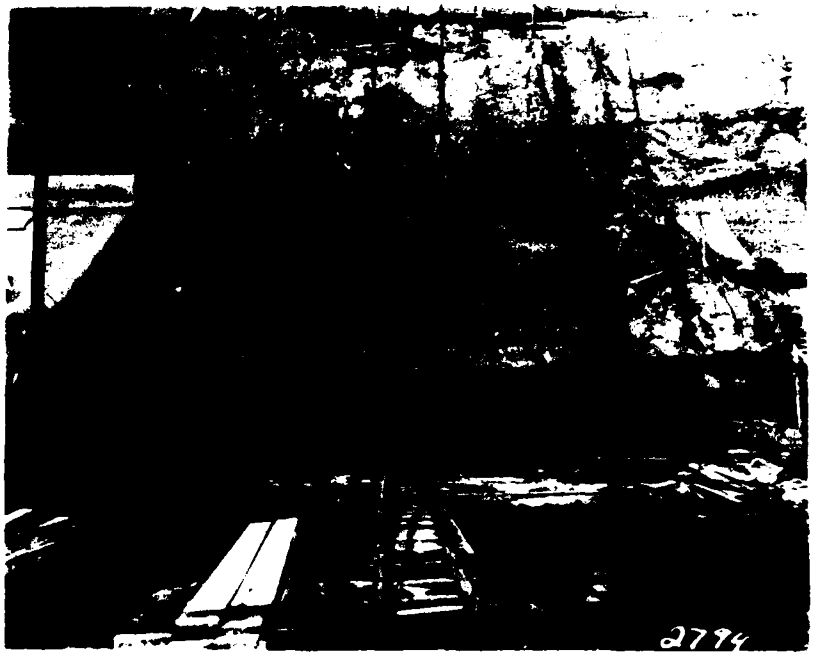

PHOTO NO. 35 View of downstream portals
showing steep slope and 1A
and BB Lignites

PHOTO NO. [illegible] [illegible] portals.

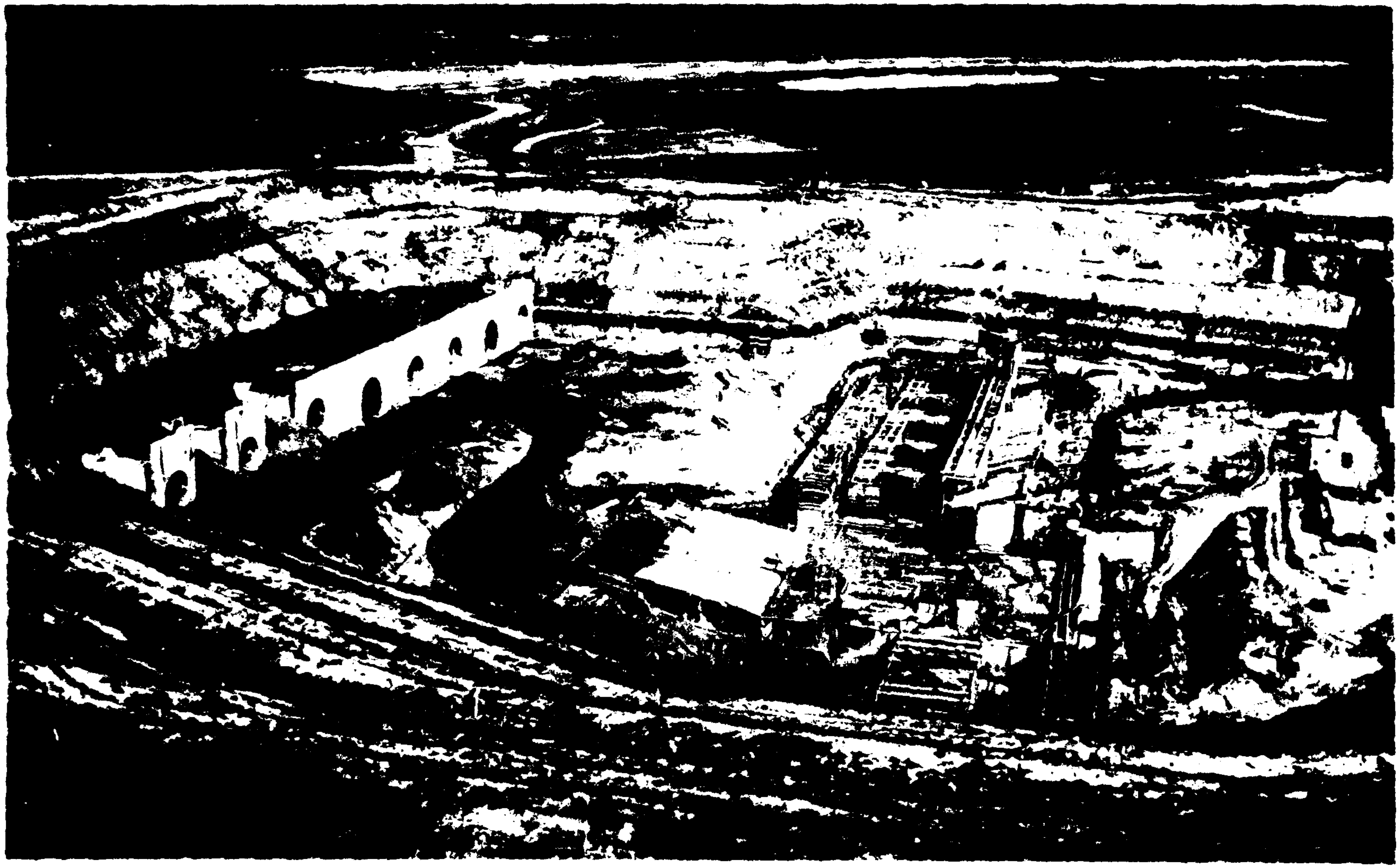

PHOTO NO. 37 View of surge tank, powerhouse and stilling basin area

[illegible]stream portal area

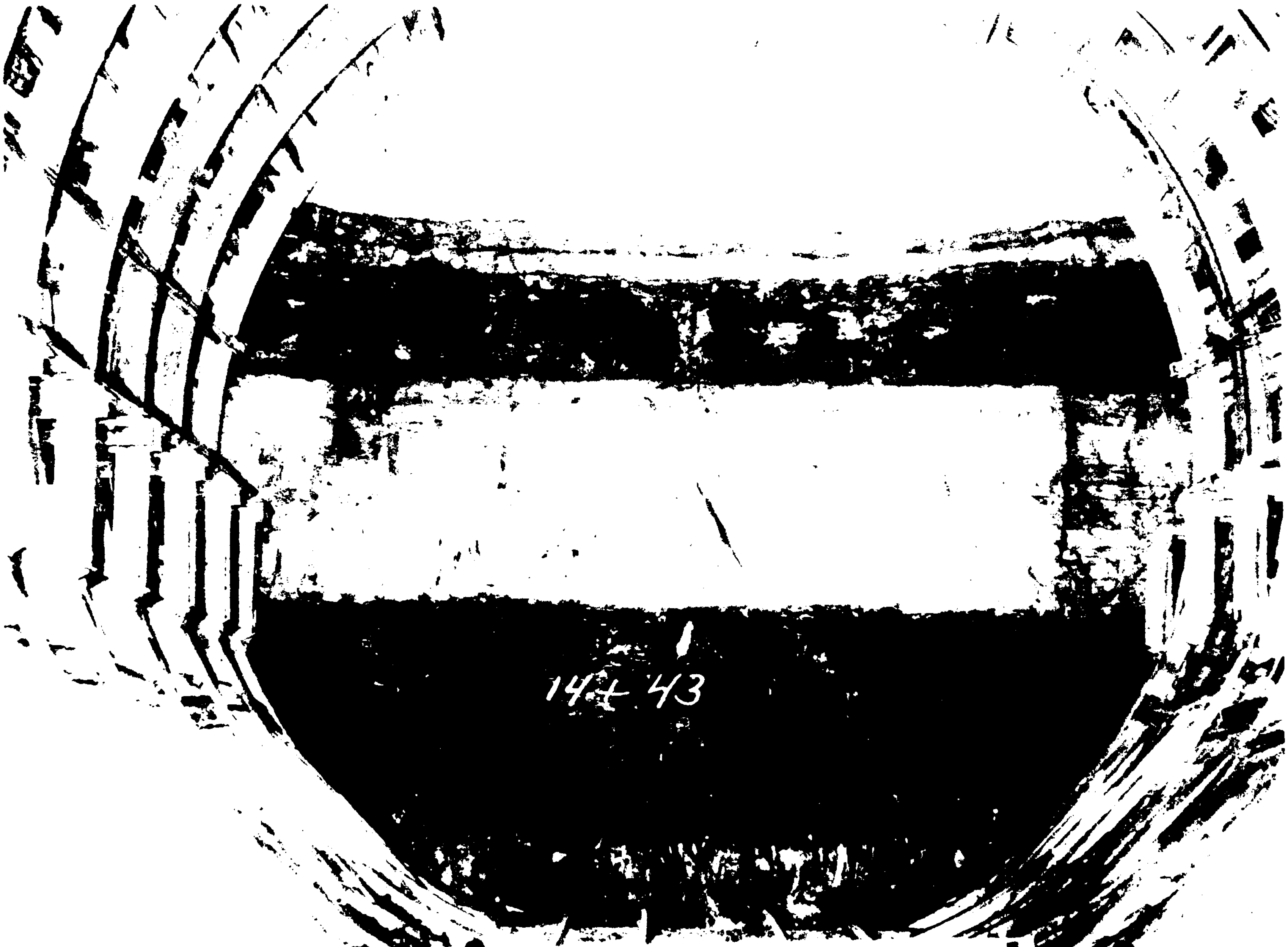

PHOTO NO. 39. View of Fort Union in tunnel heading.

PHOTO NO. 41 View of intake bridge pier 1 foundation

PHOTO NO. 42 View of intake bridge pier 2 foundation

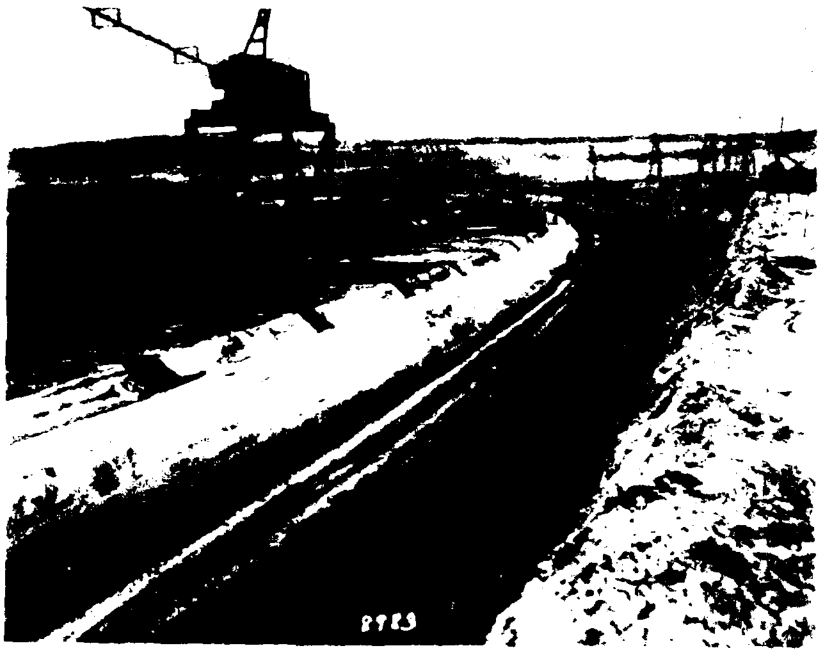

PHOTO NO. 43 View of spillway crest key from east

PHOTO NO. 44 View of spillway crest key from west

PHOTO NO. 45. View of spillway chute looking upstream towards crest structure.

PHOTO NO. 45. View of spillway chute looking upstream towards crest structure.

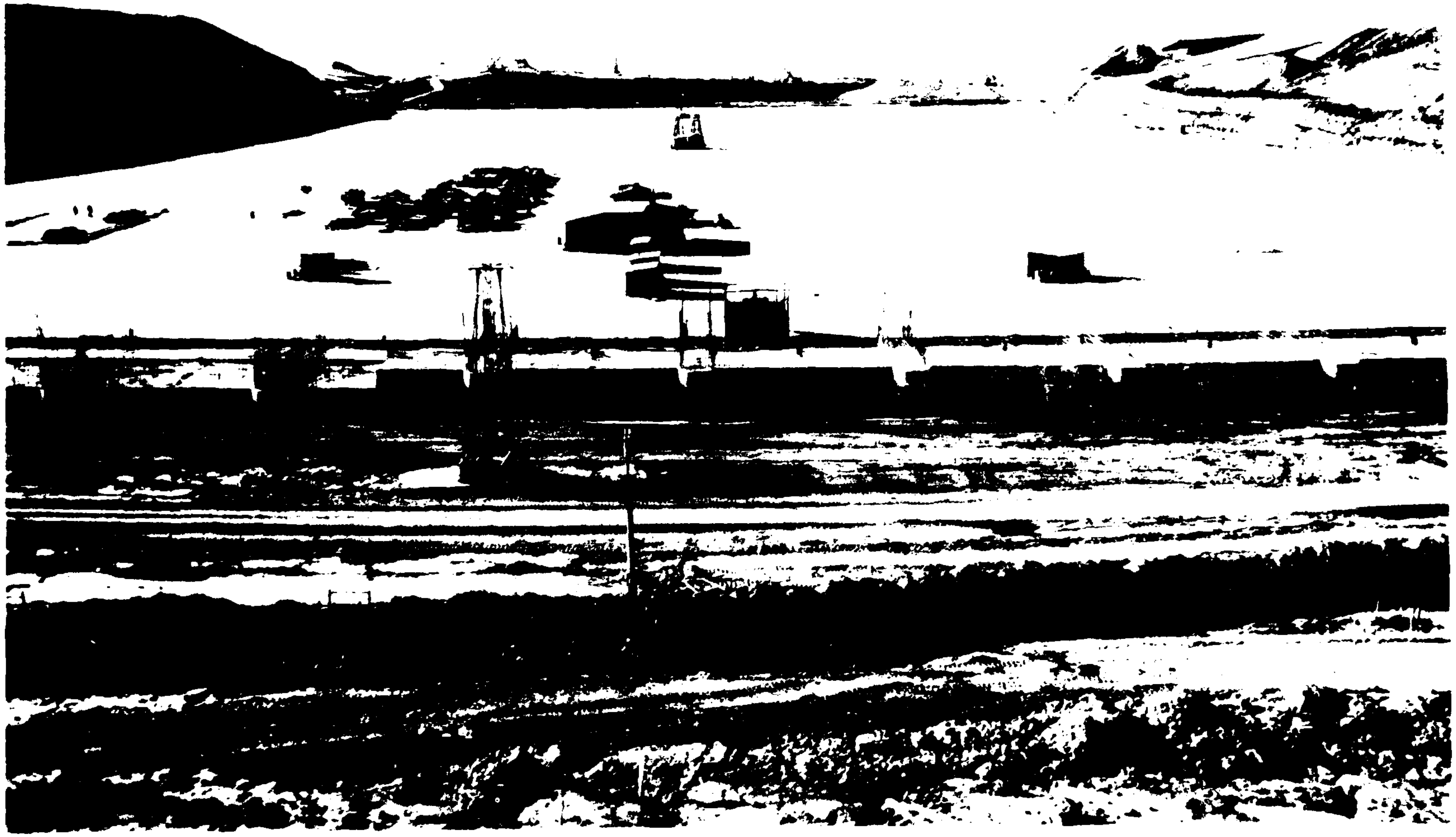

PHOTO NO. 46. View of spillway chute looking downstream with crest structure in foreground.

PHOTO NO. 47 View of typical chute wall foundation

PHOTO NO. 48 View of typical lateral drain excavation

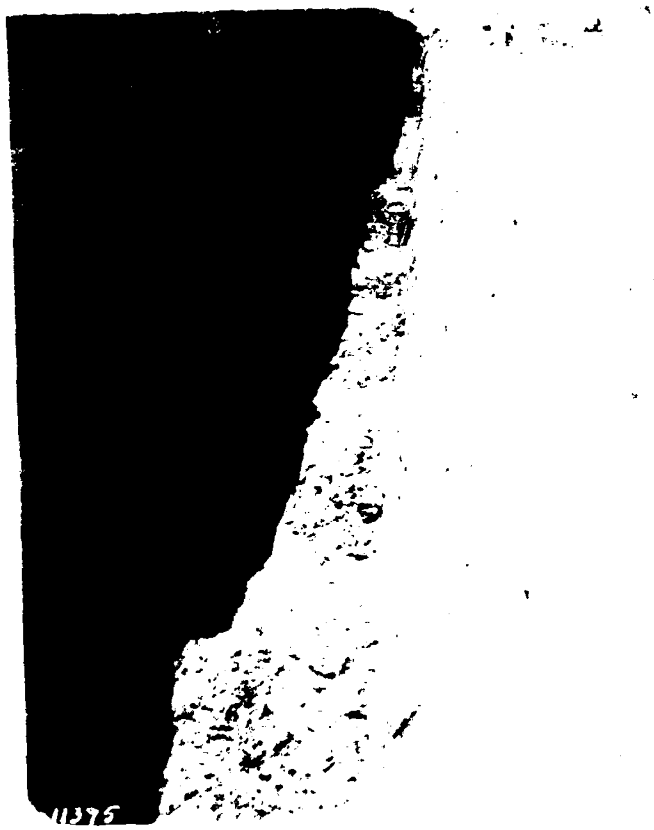

PHOTO NO. 49 View of overbreak of Fort Union Clay Shale in stilling basin end sill

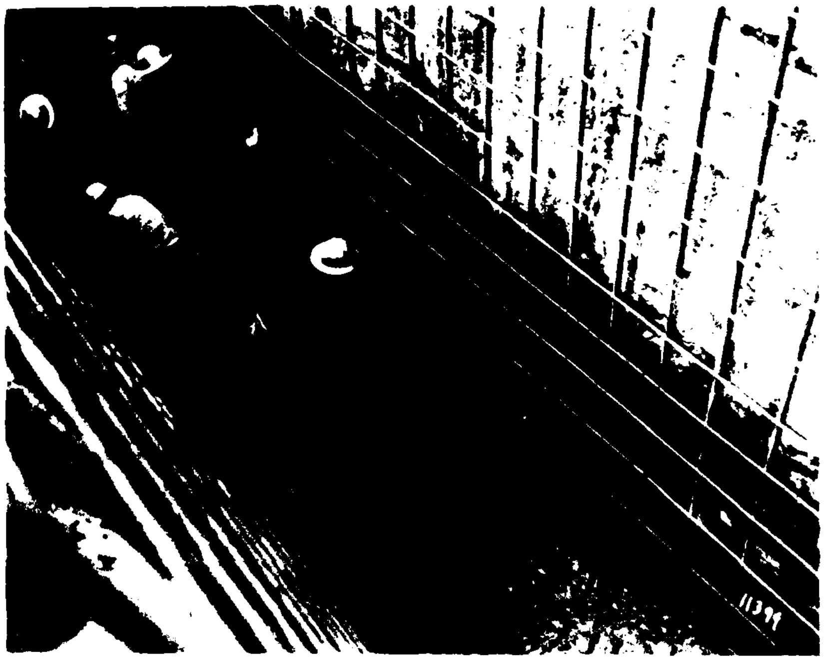

PHOTO NO. 50 View of excavation for end sill foundation prior to clean-up

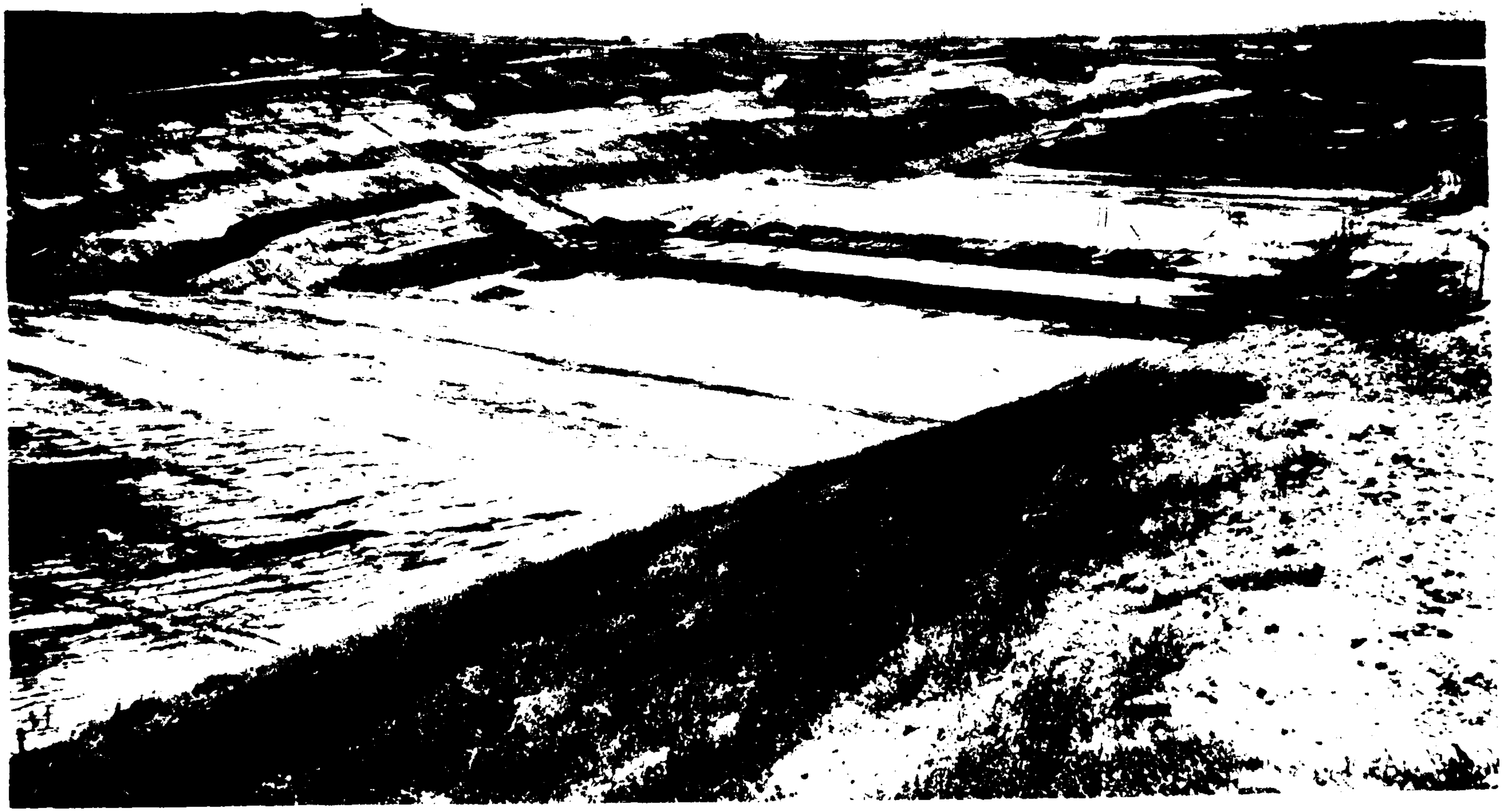

PHOTO NO. 51 View of spillway stilling basin foundation.

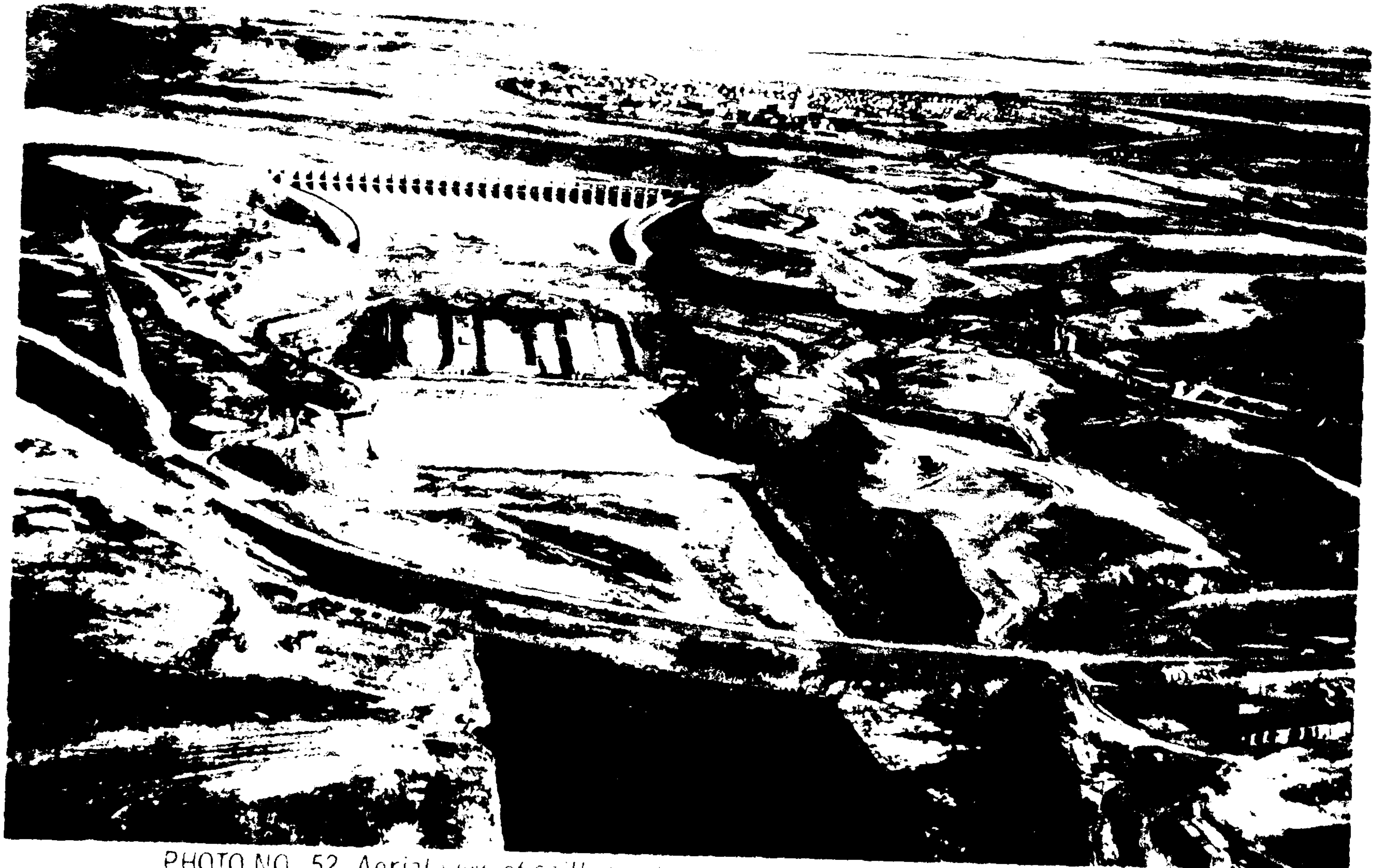

PHOTO NO. 52 Aerial view of spillway showing approach channel, crest structure, stilling basin and discharge channel.

PHOTO NO. 54 View of initial embankment construction during Stage 1 earthwork.

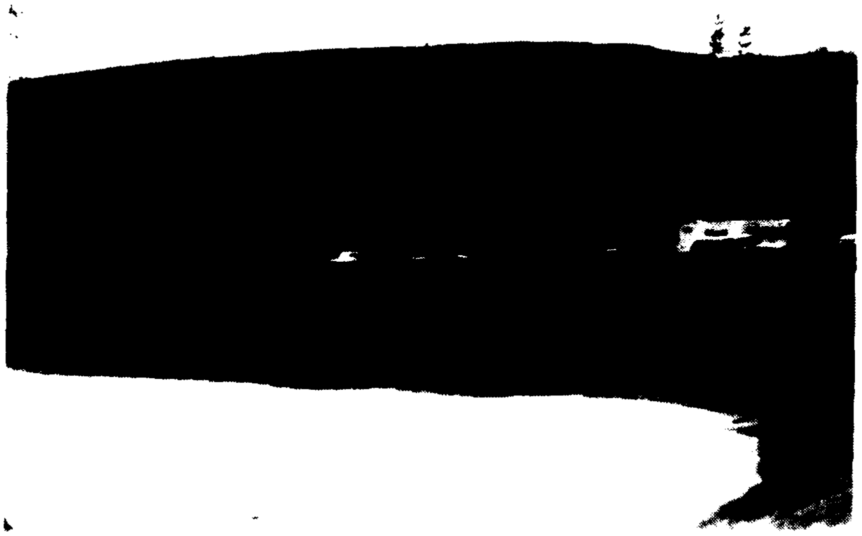

PHOTO NO. 55 View of surface slide west slope of spillway

PHOTO NO. 56 Aerial view of Snake Creek embankment

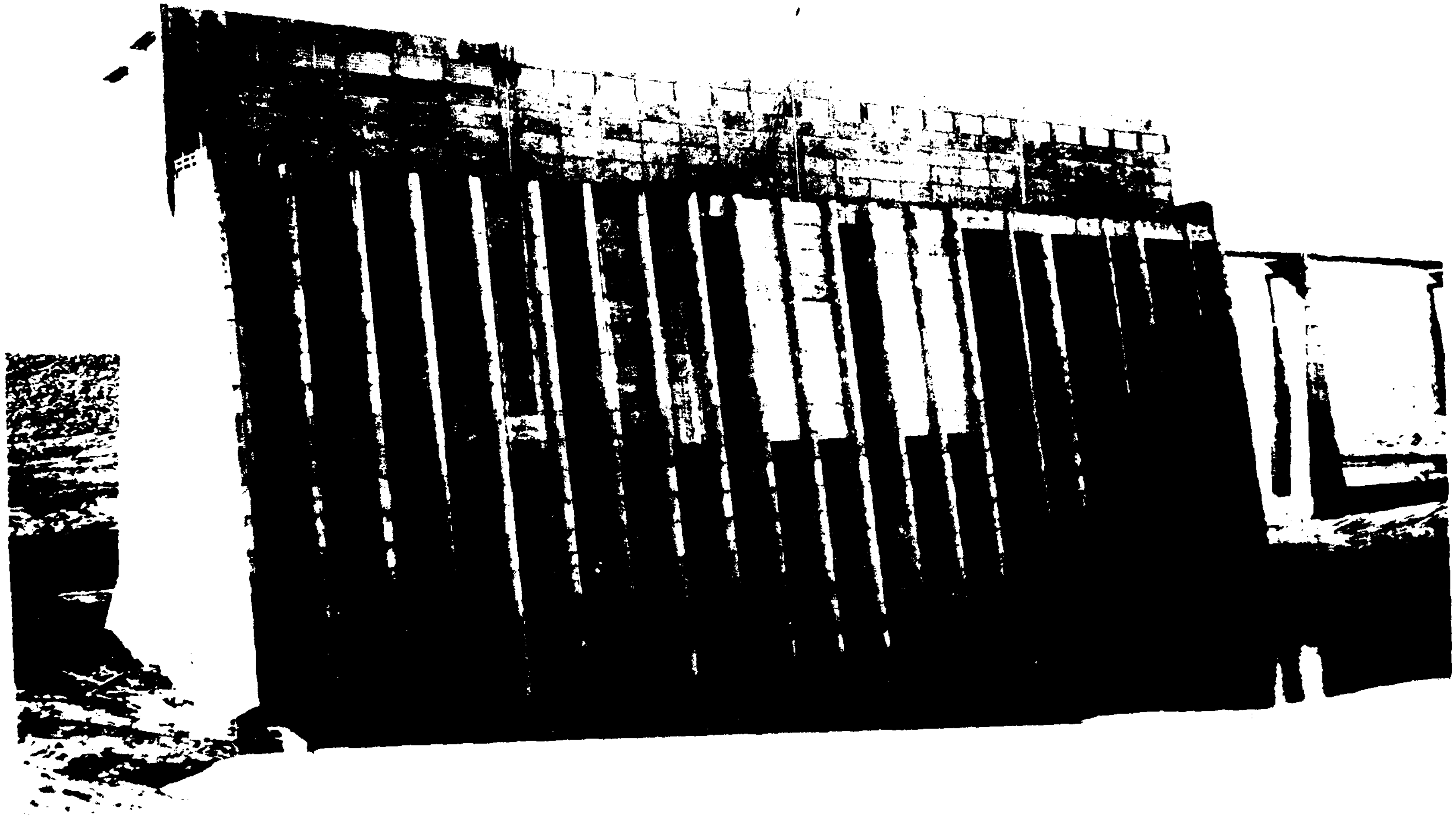

PHOTO NO. 57. View of finished intake structure.

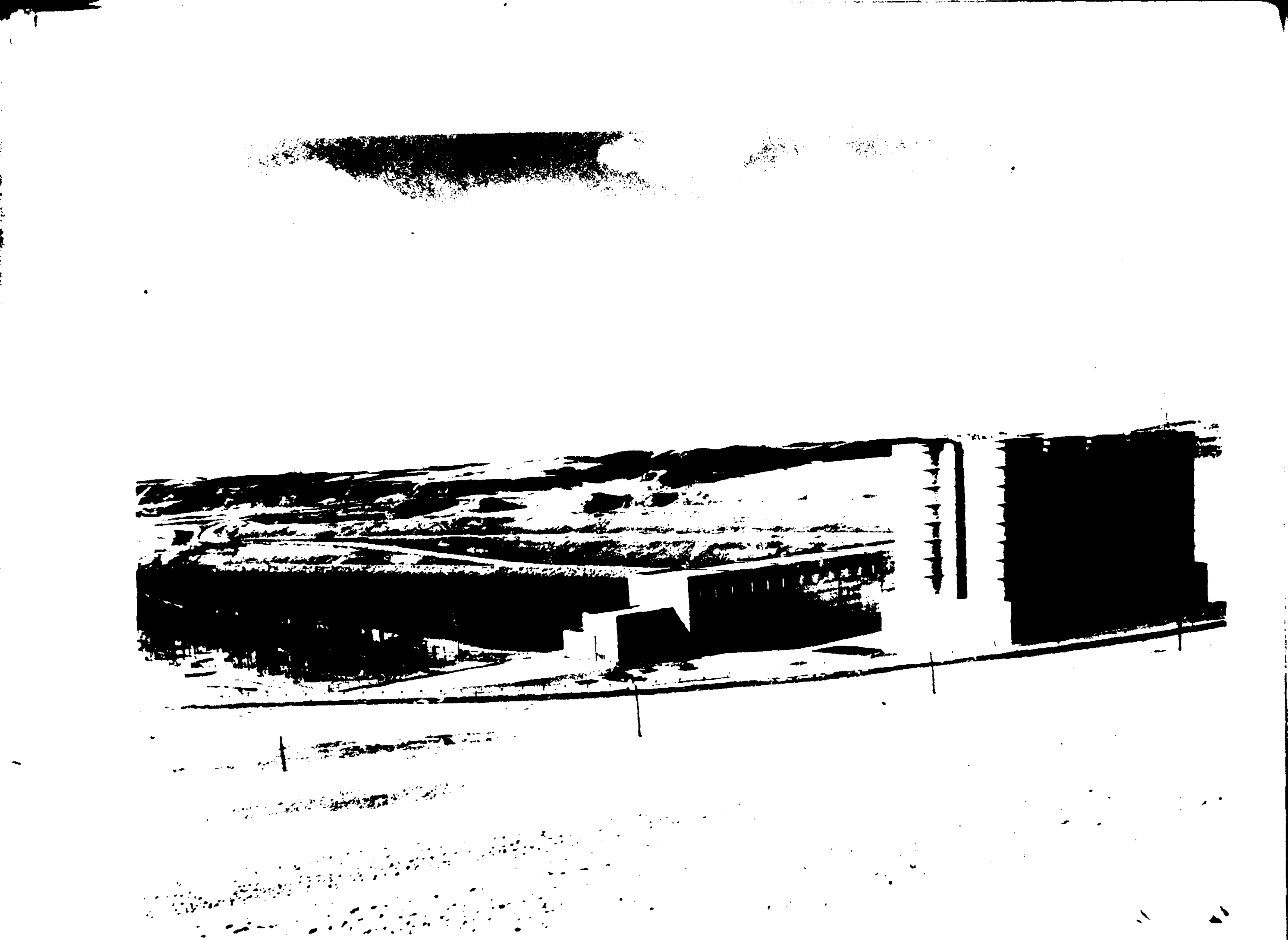

PHOTO NO. 58. View of completed powerhouse and surge tanks.

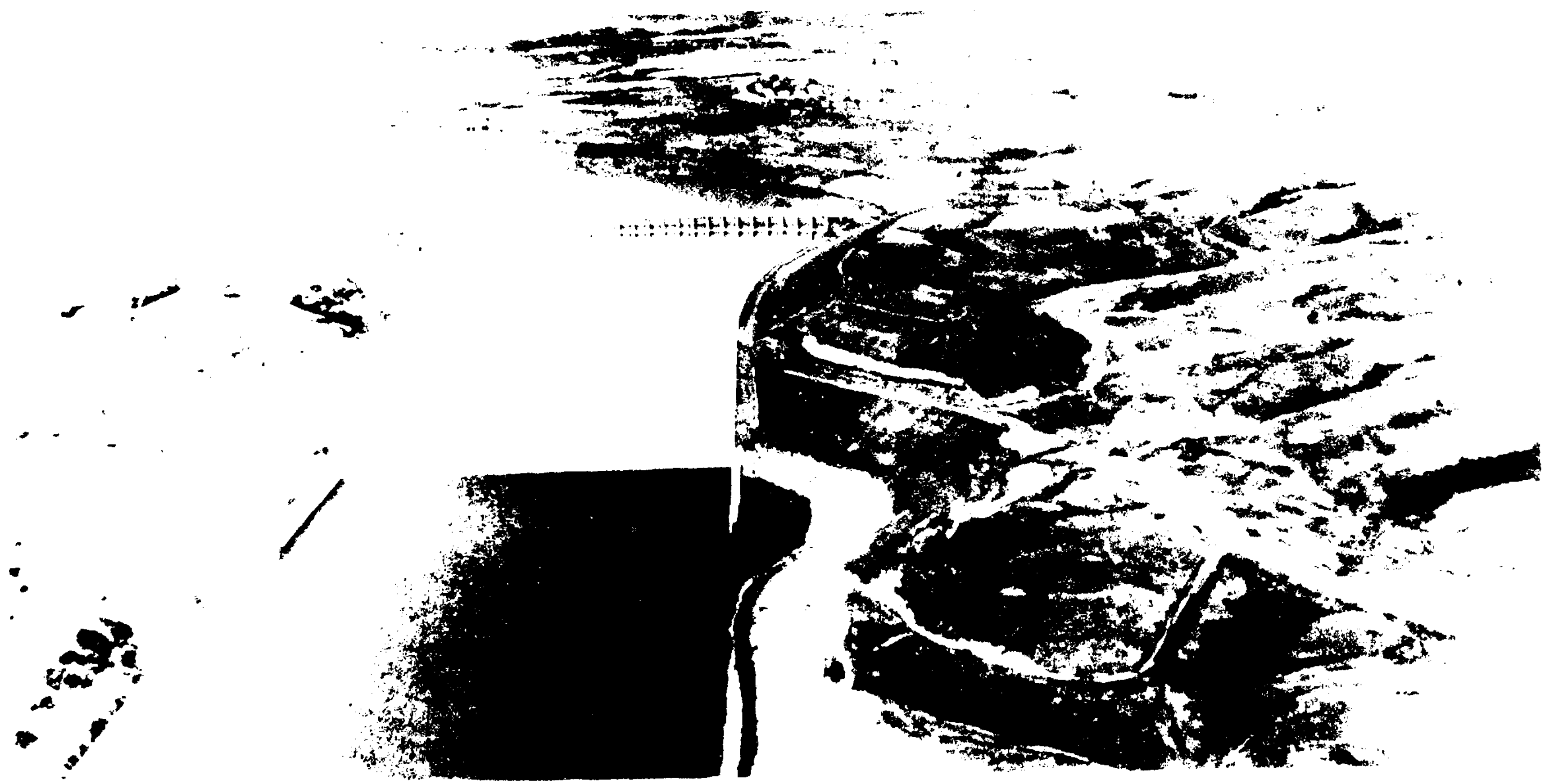

PHOTO NO. 59 View of completed spillway.

PHOTO NO. 60 Aerial view of completed embankment.

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